ENGINEERING RECORD

A WEEKLY JOURNAL DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

PUBLISHED BY THE McGRAW PUBLISHING COMPANY, INC.

Volume 68 NEW YORK, SATURDAY, AUGUST 9, 1913 Number 6

Efficiency Kinks for Work Trains At the present time numerous railroads are doing much of their construction work with their own forces and equipment, and are doing it on a much larger scale than that of the Louisville & Nashville work described on page 162. The latter work is of interest because it is a new departure for this company, because careful cost records embracing all items of expense are being kept on the entire work, and because much attention is being given to the perfecting of details of equipment and operation. In the case of the steam shovel, alterations and additions have been made frequently, and in its final state, with the large locomotive tender and the workshop caboose, the outfit is capable of long-sustained effort, while all ordinary repairs can be made on the ground without loss of time. Another feature worthy of special note is the use of the extra tenders for the work locomotives, whereas with only one tender there are frequent delays while the engines are running for water. To permit the use of cheap construction track only light engines are employed and the use of a single large-capacity tender would mean too great a concentration of load. Both difficulties are therefore avoided by having two small tenders.

Getting Action on Good Roads The story of the Winona County roads published on page 159 of this issue contains two important points aside from discussion that might arise as to the advantages of the 8-ft. concrete pavement. The most striking feature of the story lies in the example of what can be done by public-spirited citizens who go at road-building in the same way as they would at building a club-house or similar private institution. They were so convinced of the advantages of a good road that they were willing to pay for it in part by personal contributions, in addition to their regular taxes. The leaders showed a desire for a good road rather than a good-roads office. This is the point that many national, state and county good-roads associations may well take to heart. No organization aiming at the sane development of highways should be laughed at; but there is no doubt that the haphazard publicity acrobatics of a leading-citizen secretary and the annual election of officers constitute too nearly the whole of the campaign and results of many organizations. The second point is the missionary work performed by a mile and a half of good roads. No matter how many tables and diagrams are published showing decreased tractive effort and lowered haulage costs per ton-mile resulting from improved highways, a few miles of roads passable in the spring and winter talk much more convincingly to the farmer. When the Winona County commissioners awoke to this fact there was no more petty political opposition to the good-roads movement.

The Continuation School Vocational education is receiving much attention throughout the land at the present time. Much of the discussion originates with and does not get beyond the school teachers, and thus is frequently confined to plans for elaborate all-day five-day-a-week trade and manual-training schools, which not only are run at an exorbitant cost per student, but cannot possibly be of service to the great number of children who must go to work at the age of fourteen. Moreover, the proposition to introduce vocational work into the primary grades, if it extends beyond some simple manual-training work, is still more unsatisfactory, because it lessens what little chance the pupil has of getting thoroughly some few fundamentals which at least awaken his hidden talents. Because of this interest and the great amount of misdirected effort—or at least misdirected talk—the paper by Professor McKibben on "Industrial Education in Berne," on page 164, is particularly timely. The continuation school, as has been pointed out frequently in these columns, is very successful in Germany and Switzerland. There is no doubt that it can be just as successful in this country. The results obtained in Wisconsin with an expenditure of only $10 per student are particularly gratifying and indicate that under a proper law the co-operation of manufacturers and a host of competent teachers can be secured without difficulty. Massachusetts, Connecticut and Indiana are giving much attention to the subject and when other states, and cities, too, realize that success in the work is not measured by money expenditures there will likely be a decided impetus to the continuation-school movement.

Tunneling in Fine Sand A short portion of the Lexington Avenue Subway in the lower part of Manhattan Borough, New York City, is now being constructed in fine sand near the surface of the ground by unusual, though not entirely new, tunneling methods. Conditions prohibiting the disturbance of the surface prevented the use of the covered-trench method, and on this account the structure was designed as a pair of single-track curved segmental cast-iron tubes, adapted to construction by the ordinary shield method, combined if necessary with air pressure as used for quicksand or submarine tunneling. This method involves slow work and the installation of costly plant having little salvage value, and the contractor, therefore, elected to employ a system, described on page 147, which combines different methods and utilizes ingenious details, several of which have been developed by him in previous successful tunneling through quicksand. The method adopted includes some of the essentials of the crown-bar and the shield system and is executed with a permanent lagging, special timbering, and a novel system of supplementary longitudinal girders. The excavation is made with a top heading drifted under a sectional-roof shield with a number of easily driven separate units. It is widened in successive operations providing for the uninterrupted support of the roof. Interchangeable sectional longitudinal beams and girders are introduced to distribute the heavy roof load, to provide against undermining and to form a continuous center core, giving great rigidity to the falsework and serving as a tie to connect the finished and unfinished portions of the work. The combination of sectional-roof shields, continuous steel crown-bars, steel heading and center girders, and the elimination of roof arches resting on wall plates and of roof supports carried directly to the tunnel invert differentiates this method of construction from the ordinary method with full shields or crown-bars and heavy timbering. Although the depth of the roof covering varies from a minimum of 10 to a maximum of 20 ft. and the material encountered is very fine, unstable sand, partly below water level, the work has been carried on up to the present writing without serious loss of ground or surface displacement.

Latin and Greek for the Engineer From all quarters there comes criticism at frequent intervals of the backwardness of the engineer in civic and public affairs. Few have attempted to assign reasons therefor, contenting themselves with pointing to the opportunity and duty of the engineer as a citizen. While no direct explanation for this condition is offered by Dr. Steinmetz in the paper on page 158, it is not difficult to draw from it conclusions that explain quite satisfactorily the attitude of the profession to public affairs. To do so, however, will draw the condemnation of all who advocate a utilitarian education—and they are legion—and also to inject another difficulty into the already troublesome problem of the education of the engineer. Whether agreeing with Dr. Steinmetz's remedy—classical education—or not, one cannot deny that there is among engineers a sad lack of that development of the intellect and broadening of the sympathies for which he pleads. Most engineers have taken their engineering studies as undergraduates and have therefore missed the broadening influence of the full collegiate education, which in the older professions—law, medicine and the ministry—is more generally regarded as a necessary preliminary to professional study. Until engineering is looked upon as a post-graduate course to the extent that law and medicine are now so regarded there will be but little hope of a general realization in engineering circles of the real education—a drawing out, a development, as the derivation of the word indicates—for which Dr. Steinmetz argues. There are engineers of the type he has in mind, but their broadening and development have come more from personal effort after college than from their scholastic work. The tendency of the engineering course, it is freely admitted, is to sharpen the wits and to develop the powers of analysis, but along narrow rather than along broad lines.

Railway Finance in a Nutshell A strange form of economic blindness seems to affect many people when they turn their eyes from their own business or domestic affairs to the operation of public-utility corporations. The necessity for a balance between income and expenditure is patent as long as they do not look beyond their own sphere, but that balance, in their minds, at least, seems to be replaced for the public utility by some Aladdin-like process which allows the railroad, the central station and the private water system to be indifferent to ordinary economic laws. These people can be brought to their senses when removed from the influence of demagogic newspapers or by news of a passed dividend on stock that they hold. The situation even as to a mighty railroad is extremely simple, but has been particularly well put by Mr. Howard Elliott, the new president of the New Haven railway, in an interview in the "New York Times Annalist." "You cannot have better transportation," he said, "at lower rates, higher wages and higher taxes, and procure at the same time the huge quantities of capital necessary to provide increased facilities. There are only two ways to get money for improvements. One way is to earn it and the other is to borrow it, and unless a railroad earns money it cannot borrow." The statement deserves wide publicity, especially in view of the persistent agitation for higher wages and the popular disinclination to pay higher rates. The erroneous attitude of mind referred to has been fostered by recent events—the increases granted through arbitration to the railway locomotive engineers and firemen and the denial two years ago of the prayer for increased rates. The recent decision of the Interstate Commerce Commission to look into the adequacy of present rates, however, should help to correct the ideas of those who have considered the railroad's purse inexhaustible no matter what the income.

Federal Permit for Power Development on the Pend d'Oreille River

It is a matter of congratulation that the Federal Government has adopted so progressive a policy in respect to water-power development on the public domain as that embodied in the permit, abstracted on page 145, issued last week to the International Power & Manufacturing Company for a very large project in the State of Washington It has long been the cry of the anti-conservationists that the government, in its effort to conserve natural resources, had virtually adopted a policy of obstruction, and in consequence valuable and useful water-power sites were lying idle. It is undoubtedly true that some former national administrations were exceedingly cautious in granting power sites upon public lands, and in some instances withheld approval of hydraulic projects on the ground that the Federal statutes did not adequately provide for the protection of the public interests. The Taft Administration, it will be remembered, seemed heartily in favor of water-power development but insisted, to the extent of veto, on compensation to the government. Recently a still more favorable view has been taken of the rights and powers of the Federal authority under the existing acts passed by Congress, and a number of hydro-electric undertakings have been sanctioned with somewhat elaborate provisions for their regulation and control by the government. A former case in point was the permit issued to the Great Falls Power Company early in the present year.

The most recent instance, however, is decidedly unique in several important particulars and establishes new precedents in Federal grants of such character. No compensation is required of the permittee for the first ten-year period of the grant, while the grant itself is for an indeterminate period, and forfeitable only for violation of the terms thereof, after due notice and hearing, or in case municipal, state or Federal authorities exercise the option to purchase, as provided. This gives the power company a very satisfactory opportunity to get on its feet financially and pass through the promotion and development stages with a minimum burden. There is no question of life of franchise involved, with its inevitable bearing on the financing, and the owners seem to be thoroughly protected in the rather improbable event of purchase of the works at their fair and reasonable value by properly constituted authorities—city, state or Federal—under due process of law. In this connection it is worthy of note that the permit itself may not be capitalized for the purposes of earning a return or profit, nor for sale of the property to the authorities just mentioned.

As for the rates of compensation paid to the government, they obviously encourage the fullest possible development of the available hydraulic power. Whether in detail they are entirely fair to both parties depends on a great many conditions, largely local in character, which this journal has not investigated. A little examination of the table of rates will show that the total annual compensation decreases as the percentage of full development increases, and likewise the total compensation falls as the rates for energy decrease, but in much faster ratio. These provisions clearly favor the development of the market for energy and its disposal at the lowest rates consistent with a reasonable return. Such an inducement for full development and low rates, which is automatic in its operation, seems highly commendable, and while the compensation to be paid to the government is perhaps high under conditions of low development and high rates, it should be remembered that the payment of compensation is waived for the first ten years, which, as a rule, constitutes the most difficult and critical period in any such enterprise. There is thus an inducement, furthermore, to obtain the largest available market for energy before the initial period expires.

Among other notable features of the grant are provisions for readjustment of its terms from time to time, for the prevention of unlawful corporate combinations or evasion of the terms of agreement, and for compliance with state laws and due recognition of the jurisdiction of local regulating bodies.

Taken as a whole, therefore, the spirit of the grant is a desire on the part of the Federal Government to promote the development and proper use of natural resources, and at the same time to deal fairly with those who risk their capital in such enterprises.

Unfairness to the Contractor (?)

Upon a first reading of the letter by Mr. Guy H. Chase on page 166 one is tempted to assent to his point of view, protesting against certain clauses which seem unfair to the contractor. And while this journal has repeatedly contended that arbitrary rulings on the part of the engineer are often unjust there are certain considerations which must modify approval of the views expressed.

If written expressions had but one definite meaning and could always be interpreted to convey to everyone the same thought or idea; and if each contractor, each "owner" and each engineer were strictly honest, the writing of contracts and specifications would be a simple matter. Unfortunately, however, contracts have to be drawn to cover practical, everyday conditions.

Municipal contract work in most cases must be awarded to the lowest "responsible" bidder, and the lowest responsible bidder is generally the lowest bidder who can furnish a satisfactory bond. Many contractors, particularly if habitually low, do not hesitate to furnish materials and work of an inferior grade and then put upon the engineer the task of deciding whether or not they are good enough. Specifications and contracts must, therefore, be written as though all contractors were of this class.

The necessity for drafting specifications and contracts so as to protect rigidly the interests of the "owner" arises also from the fact that the owner's principal function in connection with the contract is to pay for the work. This he cannot escape, and there is no question about the quality of the money which he must pay out. The contractor, on the other hand, is at liberty under any contract to furnish materials and workmanship that will barely come up to the quality specified in the contract, and has an opportunity, by exercising good judgment and buying closely, to take advantage of bargains to increase his profits. It is, therefore, necessary for the owner to have a technical representative to insist that the contractor shall furnish what the owner understood he was to receive, and this representative is the engineer. He is to all intents and purposes the agent of the "owner"; his duty is to see that the contractor does not cheat his principal.

There are in the profession many very good engineers and many of a very ordinary stamp; and the "owner" can pick out for his representative one of either class. If he is looking for lawsuits and troubles an inexperienced, or prejudiced, or partisan engineer can give him all the entertainment he can wish for; if he is looking for a full observance of the contract between himself and the contractor, and a final settlement at the close of the work without court proceedings, he can find engineers who will so direct the work and interpret the contract and specifications as to satisfy both him and the contractor of the fairness of his decisions.

Few engineers who have had large experience in the supervision of contracts would care to depart from the well-established forms adopted in good practice. Nearly every clause in standard specifications and contracts has been at one time or another tested in the courts, and he would indeed have to be a master of English who would attempt to depart from the accepted and tried forms to produce something simpler and fairer, and still steer clear of all the various provisions of the statute of frauds and other legal requirements that must be fulfilled in order to have the contract stand the test of critical examination in case of litigation.

A contract between two principals of high motives and integrity never gets into court. Numerous instances can be cited where large and important works have been carried on by contractors with no contract other than a brief memorandum of what was to be done and furnished. Good faith and a mutual understanding of what is to be done are all that should be necessary to secure for one party to the contract satisfactory results from the other party, and if this good faith is lacking it is hardly possible to prepare a written contract that will not let one or the other out of the contract on some legal technicality and permit a settlement in court on grounds of equity.

Many hardships have undoubtedly been imposed on contractors under the prevailing contract system; on the other hand, it is equally true that a large percentage of "owners" fail to get 100 per cent value in work done for them.

The tendency of the day in drawing up large contracts is to add an arbitration clause providing for the settlement of certain classes of troubles and difficulties between the contractor and the "owner." As a rule, however, such a clause does not take from the engineer the right to pass upon the quality of the workmanship and materials, or the satisfactoriness of many other particulars.

Taking all things into consideration, it is not clear how the present practice could be much improved upon, and the fact that occasional hardships are experienced does not indicate that the entire practice is mischievous or onesided.

Responsibility for Calculations of Subordinates

Few problems of the engineering executive are more serious than the securing of accurate calculations from subordinates charged with the detailed technical computations which are fundamental in the analysis of operating conditions in field and plant. There is, of course a legal side to the question, but that is not its broadest aspect. Of far greater importance in the success of an organization and to the reputation of its officers is the extent to which calculations are scrutinized prior to embodying their results in a definite policy or program of action.

In very large concerns it is obviously out of the question for the general executives to give much time to passing upon the accuracy of their subordinates' figures, but some degree of mental checking is a pretty valuable process. From first to last a keen sense of proportions is one of the most important qualifications of a successful executive in any branch of the engineering industry, and with constant practice able men accustomed to think in large figures and to check by rough unit methods often acquire an analytical skill in "sizing up" problems and situations that is almost uncanny. All down the line, from the president of a company to the head of the most humble department, one finds the best work done when, at least, approximate mental checking is regularly practised. The farther one gets from the drafting board and designing pad the briefer must be the analysis accorded each successive set of figures; but what such scrutiny lacks in minuteness it must make up in ripeness of judgment. Fortunately means are at hand through modern methods of statistical and graphical analysis, the study of unit costs and the extension of construction accounting to place in the hands of the executive reliable data upon which to base his judgments. More than this, the increasing use of the staff conference and the larger measure of co-operation required in deciding upon broad engineering plans and policies are aiding executives to preserve their judicial acumen and to acquire new and helpful points of view in dealing with matters involving vast sums of money and important interests.

In theory it sounds very well to advocate the selection of reliable men by executives and then holding the former strictly to account for all work performed; but in practical affairs the best results are not always insured in this way. If an arithmetical error made by a draftsman leads to a loss of $10,000 on a contract through inadequate checking, the situation will not be remedied by discharging the man making the initial mistake. The time to forestall such mischief is before incorrect figures get into the bid. A wide latitude is desirable for every executive and subordinate, but this does not mean that accomplished work shall be exempt from the most acute criticism which the wise seeker after industrial truth can apply. There is a great deal said in these days about outlining to those lower down in the ranks the problem to be solved and then letting them get the results in their own way—a program welcomed by every employee with a spark of initiative in his composition, but none the less to be followed by an executive appraisal competent to detect serious errors or wide departures from the probable truth. A little use of mental arithmetic or the slide rule, coupled with the cool and experienced judgment of superiors sitting upon the calculations and data singly or in conclave, as their importance dictates, and many a concern will go forward with new confidence to a larger and sounder business success than it has been able to command in the past. All the resources of modern business tend to facilitate this weighing of work coming up from lower desks; the comparative ease with which diverse mental equipments and personalities may be massed upon single problems takes the place of the individual's limited judgment and provides multiple checks upon the work of departments and employees which are valuable safeguards against radical mistakes in administration.

Development of Short-Span Railway Bridges

For lofty bridges and viaducts it is generally imperative that the spans, of whatever type, should be long of the kind, but for low bridges no such rule obtains. Although it may usually be considered that, other things being equal, the cheapest bridge has the shortest spans, this is far from universal and the selection of span lengths for a given traffic is governed by many considerations besides the first cost of the superstructure. The cost of the substructure, the requirements of navigation, the danger from flood, storm and ice, the ease and rapidity of construction, the cost of maintenance and renewals, capitalized total cost, and efficiency for increased duty are among the principal ones. Some of these impose contradictory requirements, making long spans desirable on some accounts and short spans on others. To equate them justly and evolve the most suitable structure for a long and costly bridge require keen analysis, wise discrimination and full appreciation of numerous uncertain points, plus the best designing and construction skill and experience. In view of these considerations and the remarkable structures across the Bush and Gunpowder Rivers, described on page 144, it is worth reviewing briefly the development of the short-span bridge.

Where the obstruction of the waterway by numerous piers is unobjectionable, as in many estuaries, river deltas and swampy localities, and where the water is not very deep, and the bottom not difficult, there is usually little hesitation in adopting short spans. The minimum span has generally been determined by the most satisfactory stringer or girder lengths for the structures selected. A large proportion of such bridges in this country have been built as wooden pile trestles and, later, with plate girders on pile or masonry piers, or as masonry arches. Wooden superstructures are always objectionable on account of the heavy renewals required and the constant danger of fire. Wooden substructures do not possess the massive solidity of masonry and may be insidiously damaged by the teredo and by alternate submersion and exposure to the atmosphere. Moreover, the rapidly increasing cost of timber is eliminating the original advantage of great economy. The wooden trestle bridges do not, however, involve difficult design or elaborate construction plant, and can be very easily and rapidly constructed or reconstructed in case of emergency.

Plate girders are more costly than wooden trestles, may not in many cases be quickly fabricated, require special plant and skilled labor for their erection, and are quite expensive to maintain and inspect. The safe capacity of most wooden trestles and plate girders is strictly limited to the loads for which they are designed, and is likely to diminish with age. Neither of them is adapted without special provisions to receive solid floors and ballasted tracks, nor do they provide mass to absorb impact and vibration.

Arches involving external horizontal reactions are not theoretically independently safe, their numerous large piers obstruct the waterway more than the piers for girders, and must be more carefully insured against uneven settlement. Their construction is slow and expensive. They have, however, very great strength and solidity, suiting them to fast and heavy traffic, are almost or entirely free from repairs, are convenient for ballasted track, allow variation in the position of rails, and may be designed to present an attractive and artistic appearance with little or no extra cost.

The most recent development, which has been under way for ten years, is the use of reinforced-concrete girder and slab types for short spans. Properly designed and constructed they are of great strength and solidity, require little maintenance or repairs, are well adapted to heavy traffic, and, so far as can be predicted from the limited period during which they have been in service, will prove practically indestructible. Their use under favorable conditions is, therefore, coming to be more and more considered as good practice and good economy. There are, moreover, other advantages besides those mentioned. The materials for them may be secured abundantly in nearly all localities, simple standard plant only is necessary for their construction, the dead-load is so great and the unit stress so low that the live-load can be largely increased, the first cost is reasonable and maintenance negligible, while, finally, the design may be made attractive at small extra cost.

These considerations very likely controlled the Pennsylvania in adopting the flat-slab type for the Gunpowder and Bush River bridges. Notwithstanding the fact that the river bottom consisted of deep, soft mud, necessitating costly foundations, it was found that the original cost, as well as the capitalized value of the bridge, justified the construction of very short reinforced-concrete spans, which were built as solid monolithic slabs supported on reinforced-concrete piers, only 26 ft. apart on centers, with deep pile foundations. The two bridges, close together, are of exactly the same design and are built by the same contractors and have 295 double-track spans, nearly all of them duplicates, with a combined length of more than 7700 lin. ft. The dead load and cost of concrete are reduced by large perforations in the piers, and the danger to the piles from decay and possibility of attack by the teredo is eliminated by carrying the concrete footings 6 ft. below the bottom of the river where they inclose the pile tops. The structures will contain 62,500 yd. of concrete and 4200 tons of reinforcing steel and, including the bascule span, will cost approximately $1,250,000.

The completion of these structures in less than one year will be a notable achievement, reflecting credit on the skill, wide experience and resources both of the designer and contractor who have collaborated in this important work.

Gunpowder and Bush River Bridges

Double-Track Reinforced-Concrete Structures with 295 Short Spans on Pile Foundations in Deep Mud

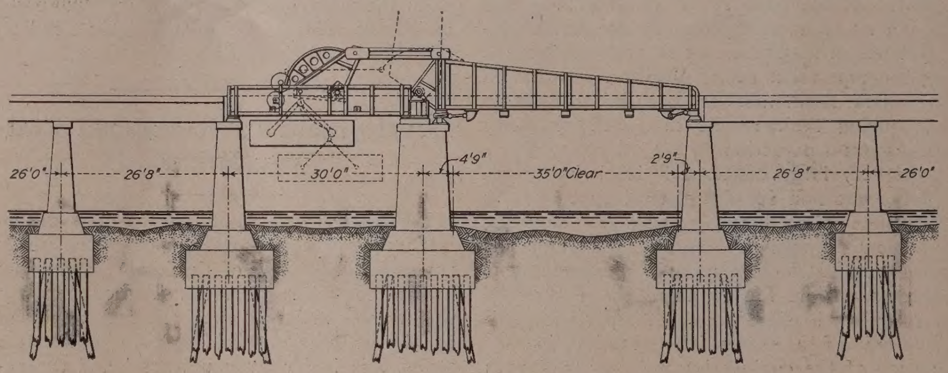
The main line of the Philadelphia, Baltimore and Washington division of the Pennsylvania Railroad crosses the Bush and Gunpowder Rivers about midway between Wilmington and Baltimore on double-track bridges, which are respectively about ½ mile and 1 mile long. The two crossings are about 7 miles apart. The structures now in service are wooden-pile trestles of ordinary type, parts of which were built nearly 60 years ago. They are not suited to the increasing traffic and involve heavy maintenance and repair charges. They are, therefore, being replaced by reinforced-concrete structures parallel and adjacent to them, and the new bridges are now in an advanced state of construction. The new structures are the longest river crossings on this railroad and are among the longest in this part of the country. The design conforms to the latest development of short-span standard construction for the Pennsylvania Railroad and is intended to develop the maximum simplicity, stability, durability and ultimate economy as compared with equivalent structures of different types and dimensions which were considered in preliminary estimates.

The two bridges are substantially alike and identical conditions prevail except that the Gunpowder River Bridge is about twice as long as the Bush River Bridge and is in a little shallower water. The details of the structures differ only in the number of regular 26-ft. spans, of which the Gunpowder River has eighteen and the Bush River ninety-nine, which together with four other spans of slightly greater lengths in each structure give them respective over-all lengths of 2751 ft. 10 in. and 4961 ft. 10 in.

Location and General Features

The Gunpowder River Bridge crosses the river about 7 miles above its mouth in Chesapeake Bay, at a point where the range of tide is about 1½ ft. and the maximum depth of water at low tide is about 4 ft. There is little current in the river, but in severe winters ice forms there to a depth of several inches, and at other times prolonged severe northwest winds drive so much water into the bay that sections of the river bottom are entirely exposed.

Although there is no dredged channel and no navigation except by very small boats, the government requirements necessitate the construction of a drawspan of 35 ft. clear width where the water is about 4 ft. deep; a single-leaf, plate-girder bascule span of 42½ ft. on centers is, therefore, provided. The bottom consists of mud underlaid by clay and sand, beneath which is gravel, generally 60 to 70 ft. below water level. The mud is exceedingly soft for a depth of 10 or 15 ft., beyond which it becomes stiffer and is mixed with sand and clay in irregular proportions and positions. The site was explored by twenty-four wash borings, about 400 ft. apart, in the middle of the river. Although these were carried down to a maximum depth of more than 100 ft., they disclosed no other materials than mud, clay, sand and gravel, which were uniformly hard, however, at the bottoms of the borings. On both old and new structures the alignments are level tangents—about 59 ft. and 102 ft. apart respectively at the two ends. The new bridge is 30 ft. 4 in. wide over all, with a base of rail 17.84 ft. above mean low water. It has one 42-ft. 6-in. and one 35-ft. plate-girder span for the bascule leaf and the operating machinery, two adjacent 26 ft. 8-in. spans, two end spans of similar length and 184 duplicate intermediate spans 26 ft. long over all. With the exception of the plate girders, all of the spans are flat reinforced-concrete slabs, cast in place in single pieces weighing about 160 tons each, and reaching from center to center of piers, where they have an expansion joint at every third span.



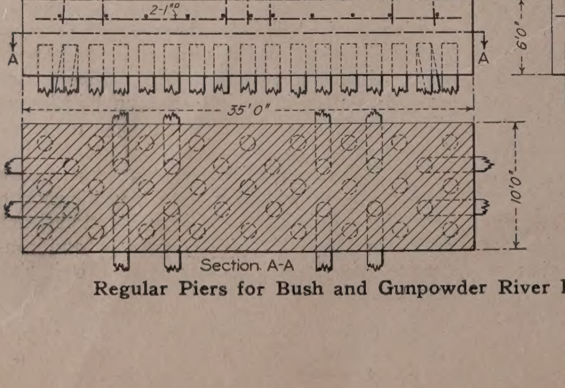
Bascule Span and Regular Slab Spans of Bush and Gunpowder River Bridges

The substructure consists of reinforced-concrete piers seated just below low-water level on concrete pedestals, with concrete footings below the bottom of the river inclosing the heads of foundation piles from 30 to 70 ft. long driven to refusal in the gravel stratum.

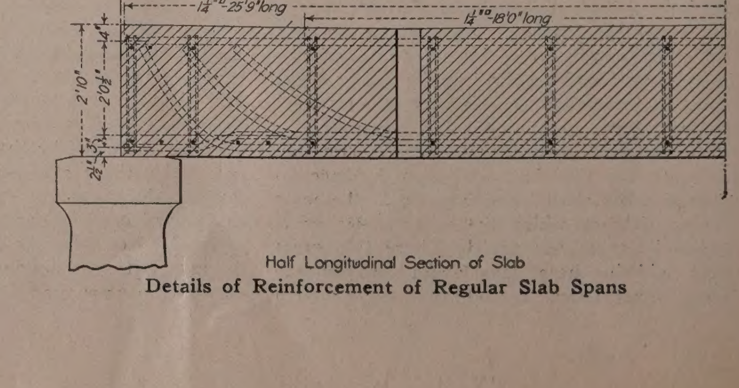
Regular Spans

Each of the one hundred and eighty-four regular spans consists simply of a flat reinforced-concrete slab 34 in. thick, 28 ft. 4 in. wide on the bottom and 26 ft. center to center of piers. Four more slabs are exactly the same, except that they vary slightly in lengths. The upper longitudinal edges of the slabs are extended with curb or fascia projections 14 in. high and 14 in. wide, cast with the main body of the slab and giving it an over-all width of 30 ft. 8 in. on top and an extreme height of 4 ft. Provision is thus made for the retaining of the 14 in. of broken-stone ballast deposited on the bridge, making the roadbed continuous with that on shore. The slabs are proportioned for two 241-ton locomotives with 35-ton axle loads, and are designed for maximum stresses of 600 and 12,000 lb. per square inch respectively in concrete and in reinforcing steel. They are made of 1:2:4 concrete mixed with Lehigh Portland cement and washed gravel ¾ in. and under. Each slab contains about 14 tons of twisted-steel reinforcement bars—chiefly 1¼ in. square, except for the ½-in. untwisted stirrups.

The original design provided for the construction of each slab in four equal longitudinal sections, permitting them to be cast in

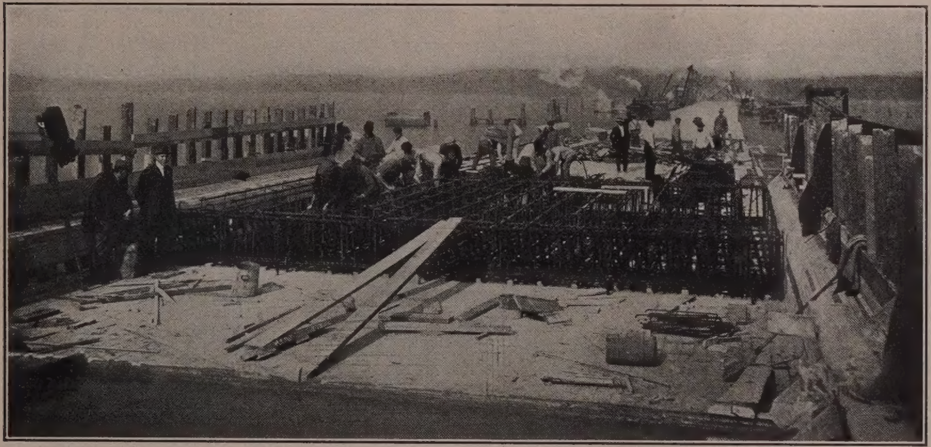


Regular Piers for Bush and Gunpowder River Bridges



Details of Reinforcement of Regular Slab Spans the contractor's yard and transported complete to the site and erected on the piers. The contractor, however, elected to cast the slabs in place on the piers, and they are, therefore, single full-width monolithic pieces, each of them making one complete span. The upper surface, 28 ft. 4 in. wide between curbs, is divided into four equal rectangles, each of which is graded slightly to the center point, where there is provided a vertical cast-iron drain pipe with the upper end covered by a grating. Small pipes are set in the curb at convenient intervals to provide for fastening the gaspipe rails and other pipes through the curb to afford connections for conduits which are to be suspended under the curb.

A space 1 in. long is left between the ends of adjacent slabs at expansion points and is filled with tar paper to make the separation of the concrete certain. Tar paper is also placed between the slabs and the tops of the piers at expansion points for the same purpose. One end of every third slab rests on the top of the pier without physical connection to it, thus affording an expansion joint. All other ends of slabs are connected to the piers by four vertical 1½-in. steel dowel pins cast in the pier concrete and engaging the concrete in the fixed ends of the slabs.



Reinforcement Steel Assembled in Place for 26-Foot Slab Span

Piers

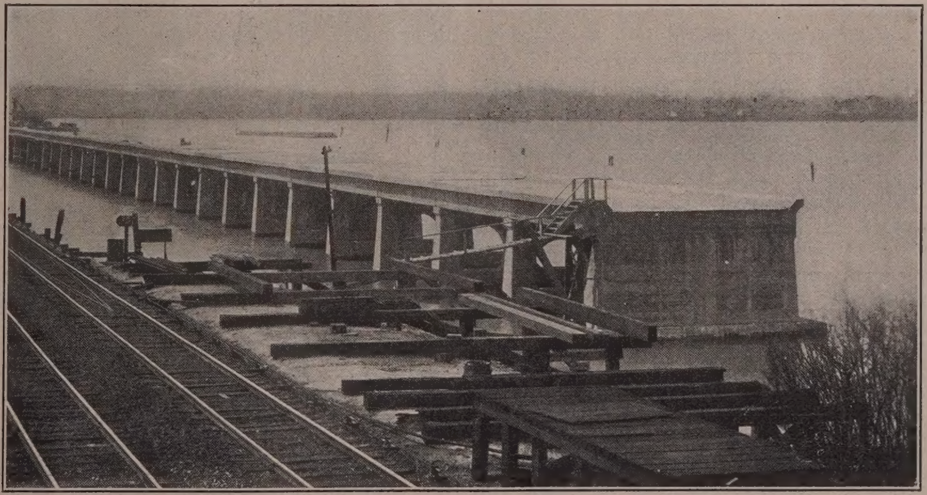
There are one hundred and eighty-six duplicate regular piers, two abutment piers, two rest piers, and one center pier—all of reinforced concrete, with pile foundations. The footings, 6 ft. deep, are made of concrete mixed nominally 1:2:4 with 1½-in. graded washed gravel and an additional 20 per cent of cement to compensate for wash due to the deposition of the concrete under water. Bottom dumping buckets were used, operated in the cofferdams before the latter were pumped out. The pile tops have an irregular penetration of from 2 to 4 ft. in the footing concrete, and the latter is reinforced above them by a crossed tier of rods laid under water. The concrete was carefully placed to bring the top of the footing to an approximately horizontal plane without dressing or leveling, and after it had set the pedestal and the pier shaft of 1:2½:5 concrete were built on it in the dry after the cofferdam had been pumped out.

The center pier carrying the bascule span is similar to the regular pier, except that it is much larger, having a footing 16½ ft. wide, supported on one hundred and thirteen piles. Just below the main opening in this pier and in the one adjacent to it, opposite the bascule span, there are two rectangular openings above water level, each of which receives the ends of five 20-in. I-beams inclosed in solid masses of concrete forming slabs 25 in. deep and 5 ft. 7 in. wide for the counterweight platforms.

The rest piers are similar to the regular piers, except that they are 13 ft. 5 in. wide and that each of them is supported on seventy-four piles. The abutment piers have 10 x 35-ft. footings and forty-three piles each; their shafts are made solid with one vertical and one battered face, and are 2 ft. thick at the top and 5 ft. thick at the base.

The concrete forms are made of dressed lumber carefully fitted. This gives a smooth and attractive surface to the concrete, no dressing or finishing treatment being required. The bridge contains about 41,400 cu. yd. of concrete and 2710 tons of reinforcement steel. The Bush River Bridge, with ninety-nine regular spans and the same special spans, contains about 23,800 cu. yd. of concrete and 1494 tons of reinforcement steel. The contract prices for the two bridges aggregate about $1,250,000.

The bridges were designed and are being constructed under the direction of the engineering department of the Pennsylvania Railroad, of which Mr. A. C. Shand is chief engineer; Mr. H. R. Leonard, engineer of bridges and buildings, and Mr. C. W. Thorn, assistant engineer in charge at the site. Henry Steers, Inc., and the Brann & Stuart Company, Inc., are the contractors.



View from Old Bridge of Partially Completed Concrete Structure

---

A 40-Ton Girder 8 ft. deep and 63 ft. long was hoisted 255 ft. above a street in New York and set over an occupied building in twelve minutes by Milliken Brothers, Inc.

Permit for Development of Power on Pend d'Oreille River

The Federal Government, through the Departments of the Interior and Agriculture, granted a permit on July 29 to the International Power & Manufacturing Company to develop power on the Pend d'Oreille River, also known as Clark's Fork, in the State of Washington. Literally the permit covers the use and occupancy of certain lands of the Department of the Interior and in the national forests, said use and occupancy, however, being necessary for the construction of the works, part of which—the dam—had already been authorized by Congressional act. The permit is, naturally, quite lengthy, but most of the clauses are either obvious or purely administrative in character. Certain provisions as to state regulation and the use of the lands in connection with any unlawful trust and an agreement that the lands and permit shall be considered as of no value in respect to sale of the property or to regulation follow closely those contained in the transmission line permit to the Great Falls Power Company, abstracted in the Engineering Record of Feb. 8, 1913, page 162. The clauses specifying the rates to be paid the government are, however, an innovation, and, therefore, are reprinted here, together with such provisions as affect them.

A statement has been given out by the Secretary of the Interior to the effect that "at lowest stages the stream-flow is sufficient for the development of about 120,000 continuous horsepower and by use of storage facilities upon the stream above 350,000 hp can eventually be developed during years of low precipitation.

Definitions

The following four, of many definitions, have a bearing on the remuneration clauses:

"Nominal stream flow" means the sum of (a) the average of the values estimated for the mean natural flow for the two-month (calendar) minimum-flow period in each successive five-year cycle or major fraction thereof, and (b) the increase in such average due to artificial means other than the project works.

"Project storage flow" means the estimated increase in nominal stream flow made practicable by the project works.

"Available stream flow" means the sum of nominal stream flow and project storage flow.

"Total capacity of the power site" means the power estimated to be available for transmission, and is determined as the continued product of (1) the factor 0.08; (2) the average effective head, in feet; (3) the available stream-flow at the intake (in second-feet and in amount not to exceed the maximum hydraulic capacity of the project works); and (4) a factor, not less than the average load factor of the power system, representing the degree of practicable utilization of the available stream-flow, and based on the extent of practicable forebay storage and the load factor of the power system.

Compensation

No compensation for the permission given will be required prior to the year 1923; but on or before the first day of February in each year, beginning with 1924, the permittee shall pay an amount calculated from the total capacity of the power site at rates per horsepower per year varying directly as the square of the average price for electric energy charged to customers and consumers of the permittee as determined in subsection (c) hereof and varying inversely as the square of the proportional development of the power site, as shown by the accompanying table:

	If the percentage of development of power site is—						
When the average price in cents per kilowatt hour charged by the permittee is as shown by this column—	Over 90	90 and over 80	80 and over 70	70 and over 60	60 and over 50	50 and over 40	40 or less
	Then the rates of compensation to the United States per horsepower per year will be as shown below						
0.2 and less	$0.05	$0.06	$0.08	$0.10	$0.14	$0.20	$0.31
0.3 and over 02	.11	.14	.18	.23	.31	.45	.70
0.4 and over 0.3	.20	.25	.31	.41	.56	.80	1.25
0.5 and over 0.4	.31	.39	.49	.64	.87	1.25	1.95
0.6 and over 0.5	.45	.56	.70	.92	1.25	1.80	2.81
0.7 and over 0.6	.61	.76	.96	1.25	1.70	2.45	3.82
0.8 and over 0.7	.80	.99	1.25	1.63	2.22	3.20	5.00
0.9 and over 0.8	1.01	1.25	1.58	2.06	2.81	4.05	6.33
1.0 and over 0.9	1.25	1.54	1.95	2.55	3.47	5.00	7.81
1.2 and over 1	1.80	2.22	2.81	3.67	5.00	7.20	11.25
1.5 and over 1.2	2.81	3.47	4.40	5.74	7.82	11.25	17.60
2 and over 1.5	5.00	6.17	7.82	10.00	13.80	20.00	31.25
3 and over 2	11.25	13.87	17.58	22.95	31.25	45.00	70.40
4 and over 3	20.00	24.70	31.25	40.80	55.60	80.00	125.00
5 and over 4	31.25	38.60	48.80	63.80	86.80	125.00	250.00
6 and over 5	45.00	55.60	70.40	91.80	125.00	180.00	281.25

(a) At any time not less than ten years after the date for the first payment or after the last revision of the rates of compensation the Secretaries of the Interior and Agriculture may review such rates after application by or notice to the permittee and impose such new rates of compensation as they may decide to be reasonable and proper, provided that such rates shall not be so increased as to result in reducing the margin of income (including appreciation in land values) from the project over proper, actual, and estimated expenses (including reasonable allowance for renewals and sinking-fund charges) to an amount which, in view of all the circumstances (including fair development expenses and working capital) and risks of the enterprise (including obsolescence, inadequacy, and supersession), is unreasonably small; but the burden of proving such unreasonableness shall rest upon the permittee.

(b) Complete development of the power site shall mean the construction of such permanent project works and the installation of such generating equipment as will provide for the full utilization of the total capacity of the power site.

Average Price for Electric Energy

(c) The average price for electric energy charged to customers and consumers of the permittee shall be determined by dividing the total actual and estimated annual receipts from the sale and disposition of electric energy by the total number of kilowatt hours generated, provided, that in determining said total annual receipts there shall be included estimated receipts for any electric energy used by the permittee at a price which shall not be less than 2 cents per kilowatt-hour, nor less than the cost per kilowatt-hour of generating, transmitting, and delivering such energy to the point of use, taking into account proper operating and maintenance expenses, fixed charges and reasonable allowances for renewals and sinking fund; and provided further, that if the permittee shall sell or dispose of electric energy to any consumer, said consumer being an association or corporation which the permittee owns or controls in whole or in part, or in which the permittee may have, hold, or control any interest, direct or indirect, by stock ownership or otherwise, the sale price per kilowatt-hour at which the aforesaid annual receipts from such energy so sold or disposed of shall be computed shall not be less than as herein provided for in the computation of estimated receipts for energy used by the permittee; and provided further, that if the permittee shall sell or dispose of electric energy to any customer, said customer being an association or corporation which the permittee owns or controls in whole or in part, or in which the permittee may have, hold, or control any interest, direct or indirect, by stock ownership or otherwise, the sale price per kilowatt-hour at which the aforesaid annual receipts from such energy so sold and disposed of shall be computed shall not be less than the price paid for such energy by the consumers thereof nor less than as herein provided for in the computation of estimated receipts for energy used by the permittee.

(d) Unless otherwise authorized by the Secretaries of the Interior and Agriculture, the maximum price at which electric energy developed by or transmitted from the power project may be disposed of to customers or consumers shall not exceed 6 cents per kilowatt-hour, and the maximum price at which such electric energy in excess of 2000 kilowatt-hours per annum with an average annual delivery of more than 35 per cent of the connected installation within the year may be disposed of to customers or consumers shall not exceed 2 cents per kilowatt-hour, said maximum price being determined by dividing the total annual charge to the purchaser by the corresponding total annual delivery to him of electric energy. In contracts with its customers the permittee shall specify the maximum price of final sale or resale and shall reserve the right to cancel any contract or agreement for sale or resale of electric energy that provides for a price in excess of such maximum. Complaint by any customer or consumer of a price paid by him in excess of such maximum price will be received by the Secretaries in case of and after the failure of his attempts to obtain satisfaction from the permittee or other parties selling electric energy under the power system, and thereupon, after notice to all interested parties, with opportunity for hearing, the Secretaries will determine whether this condition has been violated.

(e) The permittee shall at no time contract for the delivery to any one customer of electric energy in excess of 50 per cent of the total deliverable capacity of the power site; nor shall the permittee deliver to any customer or consumer or use in its own manufacturing or other operations any amount of energy in excess of 50 per cent of said deliverable capacity if and when there are pending unfilled applications for energy from other customers or consumers.

Capacity of Power Site

The total capacity of the power site shall be deemed and taken to be 112,000 hp. Said total capacity of the power site may be adjusted by the Secretaries annually to provide for increase or decrease, by storage or otherwise, of available stream-flow to an amount of 10 per cent or more, or for increase or decrease of 10 per cent or more in average effective head, or in degree of practicable utilization, and that the decision of the Secretaries shall be final as to all matters of fact upon which the calculation of the capacity or compensation depends.

Effect of Reservoirs on Dayton Flood

Because of the various reports as to conditions at the Lewistown and Loramie reservoirs during the March flood which devastated Dayton a special effort has been made by the Morgan Engineering Company, which is devising flood-prevention measures for Dayton, to secure the facts. These have now been made the subject of a preliminary report by the company and agree with the statements in the Engineering Record of April 5, 1913, page 375. The two reservoirs were built to supply water to the Miami and Erie Canal.

Description of Reservoirs and Condition During Flood

The Lewistown reservoir, the report states, has a drainage area of about 127 sq. miles and covers about 10 sq. miles in Logan County. It has an average depth of about 7 ft., and will hold 2,000,000,000 cu. ft. of water.

The Loramie reservoir has a drainage area of 70 sq. miles, a water surface of about 2000 acres, a maximum depth of 20 ft. and an approximate capacity of 600,000,000 cu. ft.

The conditions at the time of the flood were very similar at the two reservoirs. The heavy rains of March 23 and 24 raised the level of the water 3 ft. above the top of the spillway at the Lewistown reservoir, 4½ ft. above the top of the spillway at the Loramie reservoir.

At both reservoirs, the report continues, the water was almost as deep in the valley below the dam as in the reservoir above, the difference being from 3 to 5 ft. For some time before the rains which caused the flood and during the flood, the waste gates at both reservoirs were open, as is usual during the spring months. The total possible flow from all of these gates was, however, only about one-half of 1 per cent of the quantity of water that flowed through Dayton. A much larger quantity of water flowed over the spillway of the dam.

Conclusions

1. No reservoir banks were cut and no action of any kind was taken at the reservoir during the flood which would increase the high-water conditions at Dayton.

2. Two small breaks which started in the

Loramie reservoir banks were repaired within a few hours and before any damage was done.

3. Before the flood began the reservoirs were full, and after the flood had passed the reservoirs were still full, but during the flood the water was a few feet higher in the reservoirs than just below them. The conclusion is that the reservoirs did not add an inch to the height of the flood at Dayton, but that they probably held back the water to a very small extent. If there had been no reservoirs, the flood at Dayton might have been a few inches higher than it was.

4. The total amount of water in the reservoirs is so small as compared with the total amount that went by Dayton during the flood, that if both reservoir dams had failed completely and poured their waters into the Miami River, the flood at Dayton would not have been raised more than 2 ft., and perhaps not more than 6 in., though it might have lasted somewhat longer.

5. If both reservoirs had been empty when the flood began, and if both had been used to hold back flood waters and had been filled entirely full, they would not have held more than about one-tenth of the flood water which passed by Dayton, and the flood at Dayton would not have been more than a foot or two lower.

6. The reservoirs above Dayton had practically no effect of any kind upon the flood at Dayton.

Loans on Reinforced Concrete Structures and the periods granted for their repayment have been investigated by a committee of the Concrete Institute, according to the "Surveyor." It was determined that the government board had not fixed any definite period, but dealt with each case on its merits. However, the following periods are taken as a basis of comparison for the more common types of construction: Bridges with flat superstructure properly protected by a layer of asphalt or other suitable material, reservoirs and tanks containing liquid, 10 years; arched bridges protected from moisture by a layer of asphalt or other suitable material, 15 years.

Tunnel Excavation on Section 1A of the Lexington Avenue Subway in New York

Driving Single-Track Tubes with Sectional Roof Shields and Pilot Girder

Section 1A of the Lexington Avenue Subway in New York passes under Vesey Street, St. Paul's Churchyard, and the old Astor House Hotel, and connects the adjacent portions of the subway which are located in Broadway and in Church Street. The high value of the ground in this locality, the objections to cutting through the churchyard, and other considerations made the open or covered trench method of executing the work unavailable, and it was determined to build the double-track line under the Astor House, the churchyard, and structures there in two single tracks, which clear all of the existing structures.

Derrick Platform and Storage Bins over Tunnel Shaft

The tunnels have a rather complicated alignment, made up of tangents and compound curves with radii of from 190 to 300 ft. and with different degrees of grades. Each tunnel has a circular lining 20 ft. in diameter, consisting of cast-iron flange rings 20 in. long, each of them made up of ten segments and a key. They will be lined with concrete, reducing the inside diameter at the springing line to 17½ ft. The two tubes have a combined length of about 1200 ft. and involve 17,570 cu. yd. of tunnel excavation and 4355 tons of cast-iron lining.

The location is in very fine sand, containing large quantities of water up to an elevation of about 34 ft. below the surface of the ground. It is very unstable and treacherous and may be expected to be displaced for considerable distances by deep excavations, thus causing undermining of adjacent structures and necessitating an elaborate system of underpinning wherever there is opportunity for bleeding adjacent to the excavation.

Preliminary Operations

Work was commenced on Vesey Street near the middle of the section, where a 30 x 60-ft. shaft extending from building line to building line was sunk and decked over across the street to carry traffic. An elevated platform was built above it and on it were installed contractors' offices, derricks and storage bins for material hoisted from the shaft through a 6 x 20-ft. opening on the south side of the street. The shaft was sunk to a depth of about 45 ft., affording opportunity for driving the tunnel headings from it in opposite directions.

Before the excavation was commenced gas mains were laid alongside the curb on both sides of the street and the local supply was diverted to them by cutting out the existing mains under the street in order to avoid the danger of leakage through them and the possible formation of an explosive mixture with the air in the excavation. High and low pressure water pipes under the street are supported on the timbering in the shaft excavation, but provision has been made for carrying the steam mains here through an 8-in. by-pass in a wooden box elevated above the sidewalk to enable the original main to be removed while construction is in progress.

A brick sewer 6 ft. in diameter was encountered in the excavation and as fast as it was exposed the upper part was removed and a 30-in. wooden-stave pipe was constructed in it without much difficulty from the small amount of normal flow. After this was extended for the full length of the intercepted portion of the brick sewer, bulkheads were built at both ends and the flow of the brick sewer was transferred to it, permitting the invert of the brick sewer between bulkheads to be removed. The wooden sewer was supported on cradles from the transverse bracing timbers. House drains were connected to the wooden sewer, which will serve until the completion of the subway and of the new sewers alongside.

Sectional Roof Shields and Heading Driving

The tunnel excavation is made with a top heading driven under the protection of a set of five sectional roof shields or "jills," 3 ft. wide and 16 ft. 3 in. long, which are set approximately on chords of a circle with its radius 18 in. greater than that of the finished extrados of the lining. Each jill is virtually a horizontal cantilever, fulcrumed on the forward timbering bent or "set," and anchored by reaction against the tunnel roof in the rear. As the excavation progresses under it it is advanced by the operation of a 60-ton hydraulic jack engaging the rear end and reacting against blocking to the assembled timbering. (See drawing on next page.)

The straight or curved alignment on the level or inclined grades is maintained on the jills while they are driven by the manipulation of a raker brace or rocking block with the top bearing against the jill and the bottom bearing on the undisturbed earth in the floor of the heading. The procedure of driving is to first advance the jill on the inner side of the curve, bringing it over and up enough to compensate for the change in line and grade, after which the other jills are set to conform with the first. As the jills are advanced, transverse 2-in. lagging boards are set under the tail pieces and blocked up on temporary short wooden crown bars and form the jacking blocks.

When the jills have been advanced about 7 ft. the excavation is stopped and the heading is sheeted and braced by rakers to the feet of the last set. Another set is then installed 5 ft. in advance, the jills are fulcrumed on it, and jacking and excavation are resumed. The heading is about 8 ft. high in the clear, 12 ft. wide on top, and 8 ft. wide at the bottom. The inclined sides are temporarily sheeted with horizontal boards reaching from the ends of the cap to the ends of the sills. As soon as possible the crown bars supporting the transverse roof lagging are replaced by I-shape steel beams made of pairs of channels bolted together back to back with staggered joints to form continuous longitudinal girders the full length of the tunnel.

Completing the Excavation

About 15 ft. in the rear of the last bent the heading is extended to full width by removing the temporary sheeting and excavating on both sides down to the level of the bottom of the sills, retaining the earth by longitudinal lagging 5 ft. long, with overlapping ends and bearing against ribs braced by radial struts to the sills.

A narrow longitudinal trench 5 ft. deep is sheeted down on the center line of the heading to within about 50 ft. of the face and in it vertical posts are set to support the center points of the sills.

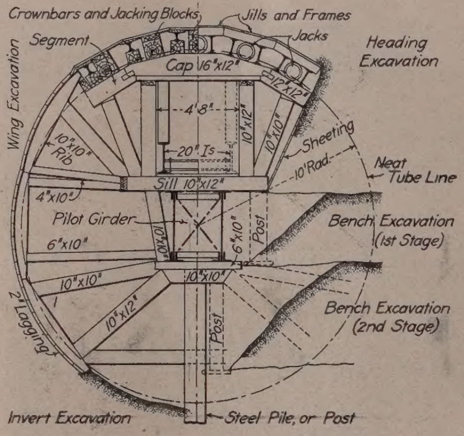
The roof load is thus distributed over several panels and provision is made against undermining the sills by subsequent operations by placing on the sills two longitudinal 20-in. I-beams against the inner surfaces of the vertical posts. These are bolted together in 22-ft. lengths to form continuous girders reaching the full length of the 5-ft. trench and support on their top flanges auxiliary posts wedged against the roof caps. Horizontal transverse 6-in. I-beams are seated on the bottom flanges of the 20-in. I-beams and are connected to the sills by vertical eyebolts. The excavation is then widened to the full width of the sills, the sides taking a natural slope, and the ends of the sills are supported by vertical posts, the center posts removed and a pilot girder installed in their place. The pilot girder, in sections 12 ft. long, is assembled concentric with the axis of the tunnel, in the trench under the sills, and the sections bolted together to make a continuous girder, designed to bind together all the timbering from the heading face to the completed tunnel lining. The roof load is transferred to it by wedging the sills against the top flanges and their overhanging extremities are braced by battered posts to the bottom flanges of the girder. The tunnel excavation is then completed down to the elevation of the bottom flanges of the girder and the earth is retained by longitudinal lagging supported on transverse ribs braced to the pilot girder.

Excavation under the pilot girder is commenced first at the splice points and afterward at points midway between them, and in dry ground is carried down nearly to subgrade, where vertical posts are set to support segmental caps bearing against the bottom flanges of the girder. Later on inclined posts or segments are set to engage the ends of the caps and permit the excavation to be extended transversely. Where ground water is encountered above subgrade, riveted sectional steel pipes are jacked down under the pilot girder to subgrade and are excavated, concreted and capped by 6-in. transverse beams, wedged up to bearing against the girder flanges.

Assembling the Lining

The cast-iron lining segments, about 6 ft. 2 in. long, 20 in. wide and 1¼ in. thick, have ribs 9 in. deep and weigh about 1500 lb. each. They are assembled in position, commencing with the invert, by a hydraulic erection machine with a radial telescopic arm commanded by a revolving gear, which, if necessary, forces them out into position and holds them there until the flange connections are bolted. After the supports of the pilot girder are transferred to the invert sections the side pieces of each ring are built up and keyed by the crown piece, the next timber set is removed, the roof load transferred to the cast iron lining by blocking, and the rear section of the pilot girder is removed and reassembled in front, and so on.

The roof channels and most of the lagging, with its supports outside of the rings, are left in position and all of the space in the excavation outside of the ring is filled with dry rubble and brick masonry and grouted after the ring joints have been calked. Finally, the rings are lined with concrete, which in the upper part extends 6 in. beyond the flanges of the segment, and is thicker at the springing line, to inclose the cable ducts, and at the invert, to allow for ballast and drainage pitch.



Sequence of Tunneling Operations

Sectional Roof Shields

Each of the five jills making up the sectional roof shield is a heavy horizontal girder that is driven into the heading over the excavation to support the roof until timbering and lagging are set, and combines some of the features of horizontal poling boards and horizontal sheet piles. It has two 12-in., 40-lb. I-beams 11 ft. long and 18 in. apart on centers, braced together by steel diaphragms. The top flanges support a 6 x 36-in. wooden cover 13 ft. long, faced on the upper side with a ⅝-in. steel plate 16 ft. 3 in. long, which projects beyond the forward end of the wood, where they are bent double around steel filling blocks and inclined slightly downward to make the driving nose. At the rear they project 3 ft. 3 in. beyond the timber to form the tail which overlaps the lagging. A brace is framed between the I-beams at the rear to receive a hydraulic jack reacting against the roof blocking already set to force the jill forward. The 60-ton jack is operated by a Simplex high-pressure hand pump connected to it by a copper pipe.

Each timber bent in the upper heading serves first as a fulcrum to support the jill and afterward to carry the lagging and roof loads. The cap consists of the segments of arch bracing with the mitered end bearings bolted together with steel scabs. The top is cut to theoretical chords subtended by the jills when the latter are set on the circle of the tunnel line; but as extreme accuracy is impossible in excavation, the cap is wedged up to bearing against the jills in whatever position they may assume. The cap is supported by vertical posts under the center section and batter posts under the end segments, all of them seated on the transverse sills 8 ft. long.

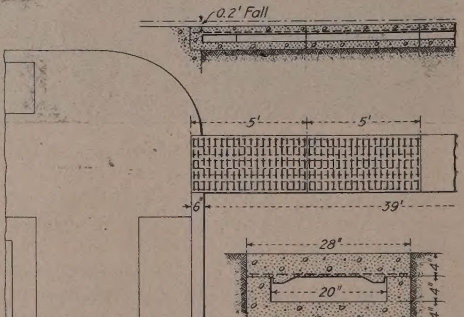
Pilot Girder

The 4 x 4-ft. pilot girder has a rectangular cross-section and is made of two lattice girders 3 ft. apart on centers, braced together by transverse and diagonal 4 x ¾-in. bars. The girder flanges are pairs of 8 x 8 x ½-in. angles, and their web members are 4 x 4 x ½-in. lattice angles. The 12-ft. sections overlap 3 ft. at the bolted joints, where extra holes are provided in the webs and flanges to permit the splices to be made at an angle corresponding with the deflection in the curve of the tunnel alignment. The roof load of the tunnel is estimated at 10 tons per linear foot, and the pilot girder is designed for a uniform load of 40 tons per linear foot on the maximum spans of 12 ft. in order to provide for uncertainty, irregularity and eccentricity of loading.

The tunnels are built under the direction of the Public Service Commission, of which Mr. Alfred Craven is chief engineer; Mr. Robert Ridgway, engineer of subway construction, and Mr. J. O. Shipman, division engineer. Frederick L. Cranford, Inc., is the contractor.

Combined Concrete Street Crossing and Culvert

Concrete street crossings in San Antonio, Tex., serving also to drain the storm water across the street, have been built on several residence streets under the direction of Mr. A. C. Pancoast, city engineer, who states that they have proved very satisfactory. The culverts are 4 in. high, 20 in. wide and are cast on the ground, continuous, half of the width of the street at a time, so as not to interfere with the traffic. The slabs, or covers, which are 4 in. thick and 5 ft. long, are reinforced with 3-in. mesh steel rib metal and easily withstand the weight of the heaviest steam rollers. From 2 to 3 ft. of rock asphalt are laid adjacent to the edges to prevent chipping. Heavy rainstorms are infrequent in this part of the country; in consequence there are few large underground sewers, the surface water flowing over the gutters and through these culverts to the creeks and the San Antonio River which course through the center of the city.



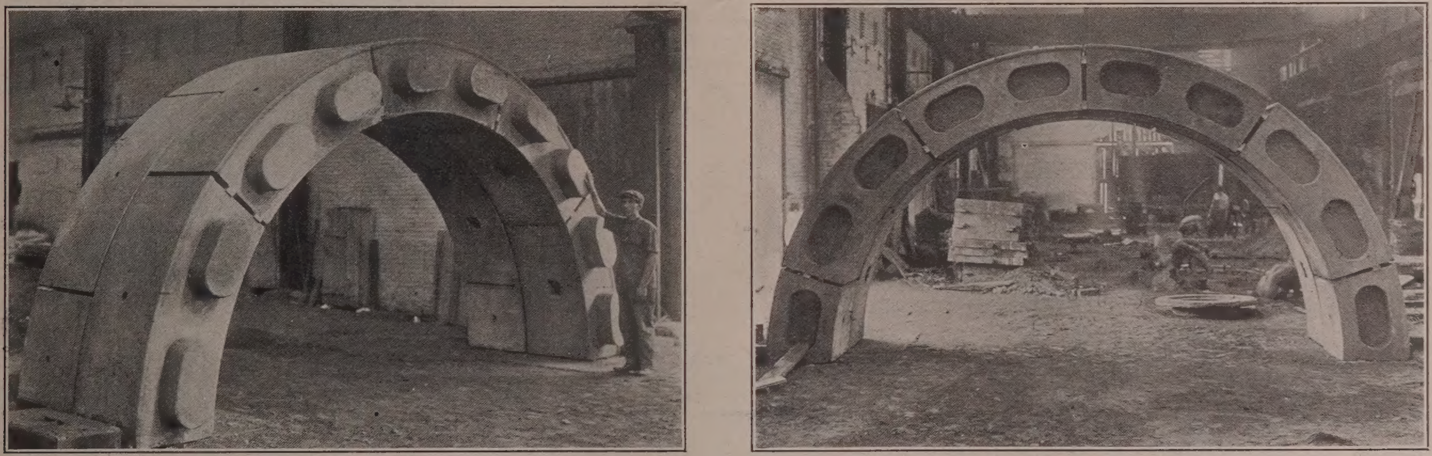
Typical Concrete Street Crossing in San Antonio

The Light-House Bureau and the Forest Service are co-operating in forestry studies on the 5500 acres of light-house reservations on the shores of the Great Lakes in Michigan and Wisconsin.

Self-Supporting Concrete Tunnel-Lining Blocks

A permanent tunnel lining of reinforced-concrete segmental blocks which is intended to eliminate timbering and afford immediate protection against a falling roof has been adopted for the Mount Royal tunnel of the Canadian Northern Railway, described in the Engineering Record of June 28, page 708. The lining has been designed by Mr. John F. O'Rourke, president of the O'Rourke Engineering Construction Company, of New York.

Substantially the same design can be modified to suit different types of tunnel cross-section, but for a circular roof arch it consists essentially of duplicate voussoirs of convenient dimensions, with plain faces for the intrados, extrados and radial joints and with two or more shoulders or key projections on one vertical face adapted to engage corresponding recesses in the opposite vertical face.



Side and Rear Views of Self-Supporting Segmental Concrete Block Tunnel Lining Erected in Shop

Erection

The first ring is erected with temporary fillers or wedges about 1 in. thick in the radial joints, and after it is assembled the voussoirs of the second, or any subsequent, ring can be assembled either from the skewbacks to the crown in the usual manner or in any succession that is found convenient, commencing at the ends, at the crown, or at any intermediate points. Since the longitudinal joints in any two adjacent rings are staggered, each voussoir overlaps two others in the adjacent ring and one of its projections engages a recess in each block in the next ring. As soon as it is assembled in place it becomes self-supporting and independent of all other voussoirs in the same ring, thus enabling the remaining segments of that ring to be erected independently in any order and be held securely in position without the use of falsework or temporary supports. When the arch ring is completed pairs of folding wedges are driven in the radial joints, keying all of the voussoirs together and developing the true arch action, which immediately provides strength to resist heavy external pressure or loads. Later both the radial and the vertical joints are grouted, entirely completing the construction of the lining and making immediately available its ultimate strength without having involved the expense or obstruction occasioned by the presence of forms or the handling of wet concrete, and without delay for the concrete to set and acquire working strength.

The vertical faces of the voussoirs have fillets in the extrados, reducing the clearance so as to retain the grout and facilitate the waterproofing of the arch ring. Molded holes in the center of the intrados of each voussoir provide for connection to the radial arm of the erection traveler. This arm is provided with fingers which enter the hole and are expanded to act like lewis bolts and grip the voussoir securely so that it may be lifted and swung into any required position by the operation of the arm. The latter is also provided with a wrist joint enabling it to be twisted as well as moved vertically, horizontally, and radially and thus set the block with quickness and precision.

Applicability

This lining is applicable to either earth or rock excavation, and under widely different conditions. The self-supporting properties of the voussoirs make it possible to erect segments of the ring at any time and to follow up the lining in accordance with various different methods of excavation. When the excavation is not completed for the arch ring voussoirs may be set for the portions completed and if necessary temporarily supported on shoring or timbers and removed as soon as the excavating for the arch is completed. The voussoirs may readily be made of very great strength to resist earth pressure or to afford protection from unsound rock and can be wedged or packed on the exterior as soon as placed. As they form the complete and permanent lining the large strength developed for preliminary service involves no additional expense.

Where the pressure is very great or the roof is particularly dangerous, either in earth or in rock, the finished arch ring may be maintained very close to the heading by excavating with a system of lower and upper headings similar to that used for the shield excavation of the Boston Harbor tunnel, which was described in the Engineering Record of Jan. 11, 1902, page 26, Nov. 7, 1903, page 552, and in several other articles. By this method parallel lower headings are driven on each side of the tunnel, and side walls of concrete, molded blocks or other satisfactory construction are built in them to carry the roof arch. A center-top heading is also driven a little in the rear of the bottom headings, and as fast as the comparatively small amount of excavation between it and the side heading is made the successive arch rings can be rapidly erected complete and put in service, finishing the lining close to the face of the top heading and providing a permanent roof under which the remainder of the excavation can be completed at leisure.

A semicircular arch with an intrados 15 ft. 6 in. in diameter has been constructed as shown in the accompanying views, with voussoirs 2 ft. thick, weighing 3750 lb. each. Each has on one vertical face two 12 x 24-in. oval projections 7¼ in. thick and on the opposite face two corresponding recesses, 6 in. deep. Each is reinforced by two sets of horizontal steel rods parallel to the tunnel axis, which are slightly convergent so as to form groups that inclose the recess at one end and at the other end extend into the oval projections near the curved surfaces, thus providing positive strength to resist shearing stresses due to the weight of the block or to any tendency for displacement between adjacent assembled voussoirs. The blocks are made of 1:2:4 concrete, with 1½-in. gravel, and are cast in cast-iron molds painted inside with powdered lead and kerosene oil, which gives them a very smooth, uniform finish. They are cast with the projections up, the concrete being poured through the collars for these projections, which are afterward closed by heavy cast-iron plates, resembling boiler manhole covers, having a slight clearance with the collars and secured by 1½-in. screws. The latter are operated to compress the wet concrete and generally displace about 300 cu. in., allowing the water to flush out and overflow, and securing a very dense block which can be stripped and handled when twenty-four hours old.

Adopted for Mount Royal Tunnel

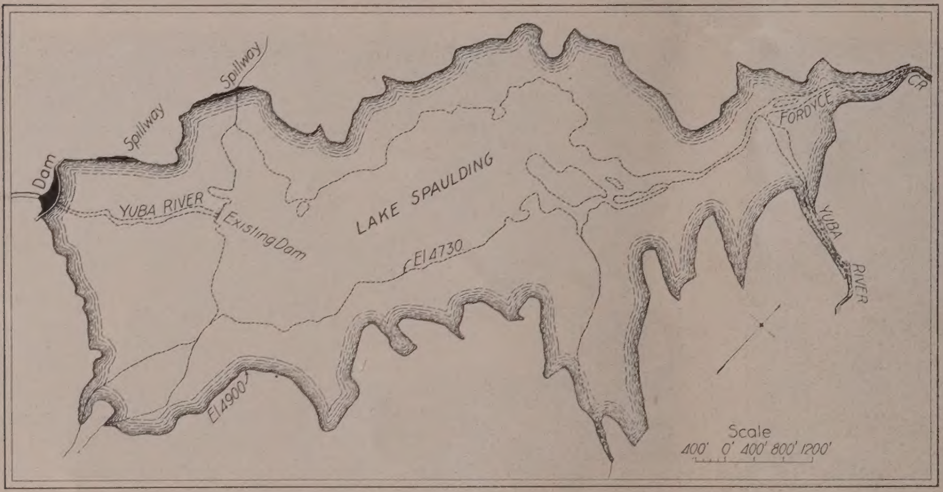
The Mount Royal tunnel in which, as previously mentioned, this type of lining will be used, is about 3½ miles long, 31 ft. wide and 21 ft. high, running chiefly through Trenton limestone and igneous rock, varying from soft to very hard, and a badly broken volcanic breccia that requires almost continuous masonry lining. Some of the limestone needs only a supporting center wall. At present the excavation is made with a bottom center heading, with breakups at frequent intervals, where the full-size tunnel section is developed and jumbo timbers are framed into the headings to maintain traffic and allow the muck to drop into the cars by gravity.

A Wash for Concrete consisting of a dilute solution of hydrochloric acid, one part of commercial 30 per cent acid to five parts of water, has been found effective in cleaning the stains from the lock walls of the Panama Canal, according to the "Canal Record." The surface of the concrete was brushed over with the solution, and afterward was rinsed with clear water. The application and subsequent rinsing freed the concrete of all discoloration. The process has proved satisfactory.

Construction of the Spaulding Dam

Design and Construction of Cyclopean Masonry Structure Which Will Eventually Be the Highest Dam in the World

A hydraulic-power development of unusual interest is now being rushed to completion by the Pacific Gas & Electric Company in Nevada County, California. This involves the construction of the Spaulding Dam, which will be the highest in the world, six power houses and two additional long-distance transmission lines, together with 54 miles of canal, of which an aggregate of more than 2½ miles will be in tunnel. In all 133,500 kw will be developed. After utilizing the water for power development it will be distributed for irrigation and will provide amply for 30,000 acres.



Present and Future Lake Spaulding and Location of Dams

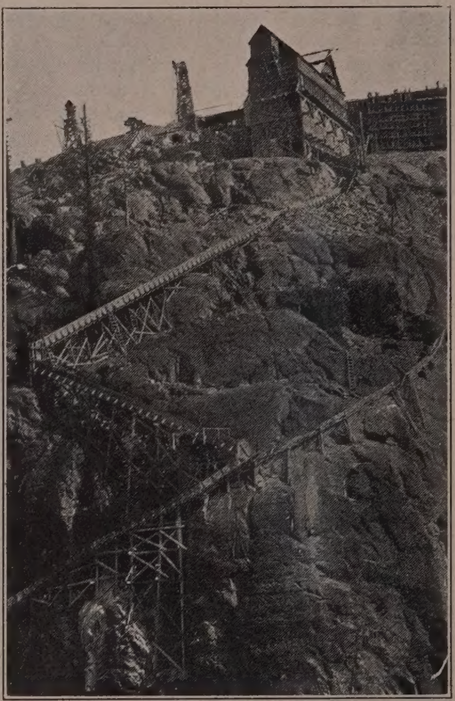
The Spaulding Dam is being constructed on the south fork of the Yuba River, about 1½ miles from Smart, a station on the Southern Pacific Railroad. This river carrries the discharge from a drainage basin of 120 sq. miles. The average annual rainfall over this area is 60 in. with a runoff of about 80 per cent. A portion of this reservoir site is already occupied by Lake Spaulding, impounded by a dam some 80 ft. in height. The new dam is being constructed across the canyon of the south fork of the Yuba, about 2200 ft. downstream from the present dam. At the dam site the geological formation is of granite and diabase, affording an excellent foundation.

Design of Dam

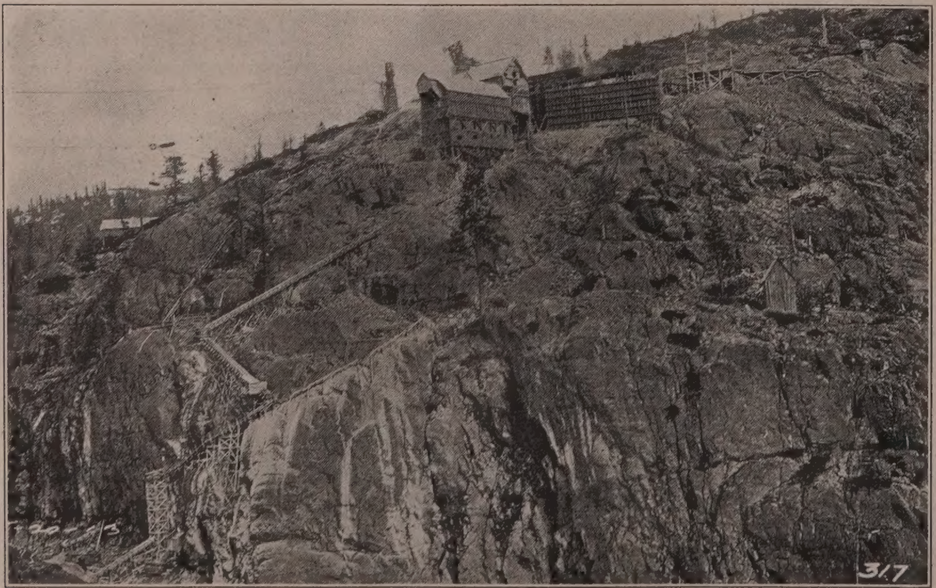
The dam itself is of cyclopean masonry. Although designed as a straight, gravity-section dam, it is arched in plan to provide an additional factor of safety.

This arch is of the type developed by Mr. L. Jorgensen, of the engineering firm of F. G. Baum & Company, of San Francisco, the main feature of which is that the angle subtended by the arc under pressure is kept as nearly constant as possible by varying the radius of the upstream side at different elevations. This is considered a very desirable condition, since arch action in the dam can thus be utilized to a maximum degree at every elevation, which tends greatly toward economy of section. When an ordinary arched dam is located in a canyon, which is narrow at the bottom and much wider at the crest, practically no arch action takes place for a considerable distance vertically above the toe. This is obvious, for the reason that near the bottom of the dam the angle subtended by the arc under pressure is so small that the arc is practically a straight line. Therefore this portion of the dam acts as a beam rather than as an arch. By using a shorter radius at the bottom, however, arch action can be utilized where it is most essential, namely, near the base of the dam, where the pressure is a maximum.

During 1913 the dam will be constructed to a height of 260 ft. above the bed of the river, but it is designed so that later it can be raised to a height of 305 ft., or 325 ft. above the lowest point of the foundation. Details of the structure to the 260-ft. level are shown in the drawing. When the height is raised the section will be thickened. The yardage of the structure after raising will be 320,000.



Concrete Chutes on Sidehill



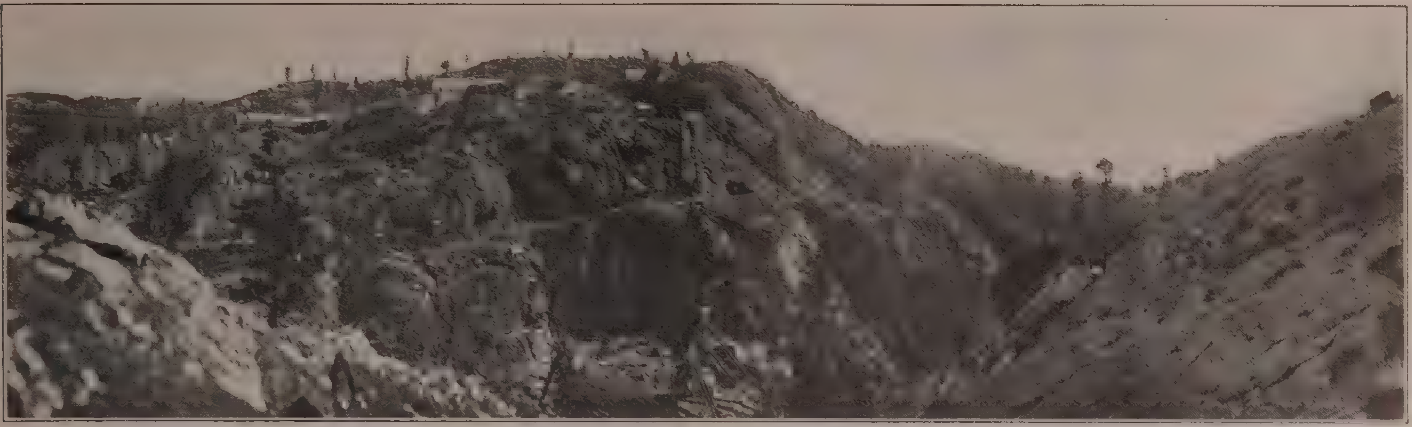
Concrete-Mixing Plant with Gravity Distributing Chutes

The reservoir will at the 305-ft. level contain 30,000,000,000 gal. of water, and a discharge of more than 350 sec.-ft. will be obtained.

A 30-in. sluice pipe of ⅜-in. steel plates extends from the face of the dam for a distance of 43 ft., where it terminates in a sluice tunnel 6½ ft. in diameter extending through the remainder of the section on a slope of 1½ per cent. A 30-in. gate valve controls the discharge of this sluice pipe. Connecting with the center sluice tunnel, and at right angles to it, is a drainage and inspection tunnel, 7 ft. wide and 9 ft. high, running longitudinally through the dam and sloping on either side toward the center. Steps having 10-in. risers and 17-in. treads are provided where necessary.

Accumulation of water between the dam and its foundation will be prevented by 6-in. weep holes, extending from the foundation vertically to the inspection gallery. A similar set of weepers is provided above the gallery, so as to collect possible seepage in the body of the dam.

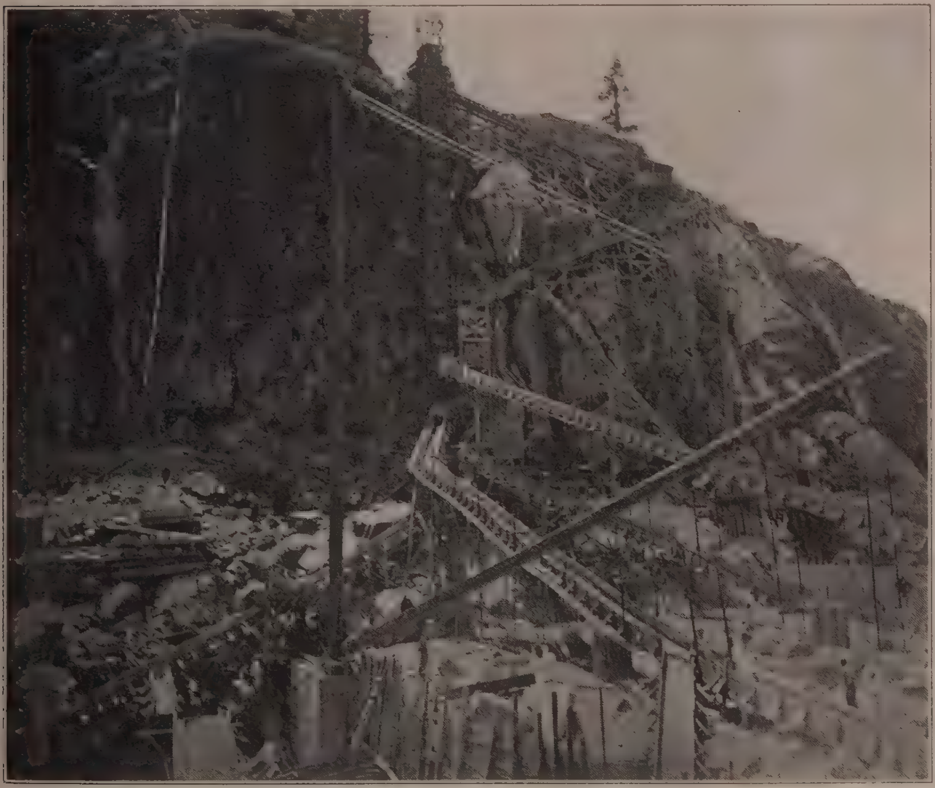
Water will be drawn from the reservoir through two 6 x 9-ft. intake tunnels opening in the granite canyon side about 12 ft. from the face of the dam. The elevations of the tunnel inverts above the level of the toe of the dam are 71.5 and 170 ft. respectively. Each tunnel is controlled by a separate gate valve, so that water may be drawn from either level. From these gates the tunnels lead to a shaft 8 ft. square, and from this shaft a single con-



View of Spaulding Dam Site, with Temporary Structure on Top of Bluff and Concrete Chutes on Slope of Hill

crete-lined tunnel, 8 ft. 8 in. in diameter in the clear and 4456 ft. long, with a fall of 7 ft. per mile, leads to the main canal.

There will be two concrete spillways, entirely distinct from the dam, located in adjoining canyons, as shown in the plan.



Dam Foundation, with Three Lines of Chutes for Distributing Concrete

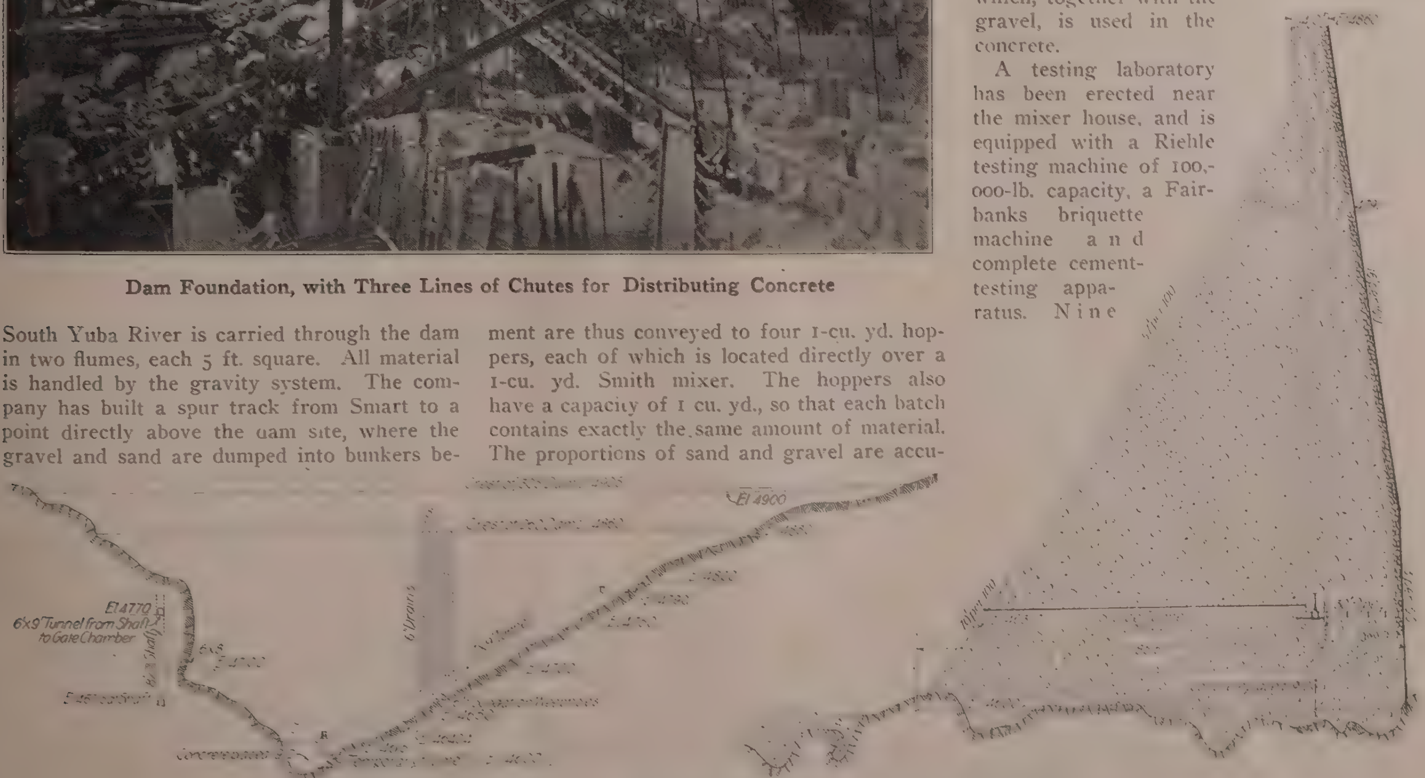
During the construction the flow of the South Yuba River is carried through the dam in two flumes, each 5 ft. square. All material is handled by the gravity system. The company has built a spur track from Smart to a point directly above the dam site, where the gravel and sand are dumped into bunkers below the cars and from there are brought to the mixers by horizontal belt conveyors, each operated by a 50-hp motor. These belts are 18 and 24 in. wide and travel at the rate of 300 ft. per minute. The cement is unloaded from the cars to a similar belt traveling at the rate of 100 ft. per minute. Gravel, sand and cement are thus conveyed to four 1-cu. yd. hoppers, each of which is located directly over a 1-cu. yd. Smith mixer. The hoppers also have a capacity of 1 cu. yd., so that each batch contains exactly the same amount of material. The proportions of sand and gravel are accurately gaged as the belts pass over a movable tripper, which directs the material into any one of the four hoppers desired. A 1:2:4 mix is used. Each mixer is driven by a 10-hp motor. The combined capacity of the four mixers is 2000 cu. yd. in twenty-four hours.

The mixers empty their contents into a chute 2½ ft. deep and 2 ft. wide which zigzags down the side of the canyon to the dam, each section having an incline of about 25 deg. The lower sections are movable, so that concrete can be placed wherever wanted. This system will be employed until the dam reaches a height of about 225 ft., when an elevator must be erected and the concrete raised from the mixers to a sufficient height to enable it to be placed by gravity even at the highest point.

After the concrete has been deposited and while it is still plastic large pudding stones are dropped into it. These are handled by three derricks, which will be raised from time to time as construction progresses and the height of the dam increases.

To guard against delay in the delivery of gravel a large storage pile is kept near the Southern Pacific tracks, while a crusher which handles the material excavated from the dam site furnishes 350 cu. yd. of rock per day, which, together with the gravel, is used in the concrete.

A testing laboratory has been erected near the mixer house, and is equipped with a Riehle testing machine of 100,000-lb. capacity, a Fairbanks briquette machine and complete cement-testing apparatus. Nine



Profile and Cross-Section of Dam, with Weepers, Shaft and Drainage Gallery concrete cylinders are taken each day and a set of three of these is tested in compression after seven days, twenty-eight days and ninety days respectively.

All the lumber so far used has been cut on or near the reservoir site and handled at the company's mill on the shore of the present Lake Spaulding. From this mill it is hauled about 1 mile over a narrow-gage railroad to a point near the concrete mixers. A saving of about $8 per 1000 ft. b. m. is effected by milling the timber on the reservoir site instead of purchasing it in the open market.

From the lumber pile near the mixers the timber is handled by two cableways spanning the dam site, each operated by a 112-hp motor. Each cableway can safely handle a load of 5 tons.

On July 5 the dam had been raised to a height of 36 ft. above the toe. By Dec. 5 the engineers of the company predict that it will be completed to the 260-ft. level.

The Pacific Gas & Electric Company is doing the work by day labor, and maintains an excellently equipped construction camp near the dam site.

Personnel

The complete Spaulding project is under the direction of Mr. F. G. Baum, chief engineer. Mr. H. C. Vensano is structural engineer and Mr. D. H. Duncanson general superintendent. Mr. G. H. Burnside is superintendent in charge of construction of Spaulding Dam and Mr. Paul Magerstadt is the resident engineer.

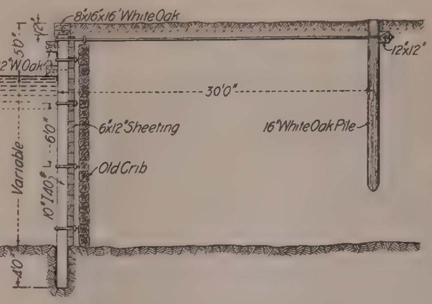
To Mr. John A. Britton, vice-president and general manager of the company, is due much of the credit for the undertaking and efficient prosecution of this work.

Reconstruction of a Timber Crib Dock at Erie, Pennsylvania

Reed dock is situated at the foot of Sassafras Street in the harbor of Erie, Pa. When purchased by its present owner, the Erie Sand & Gravel Company, it was rectangular in shape, 700 ft. in length by 122 ft. in width. The south side of the rectangle was the shore line of the harbor, and its west, north and east sides were timber cribs of irregular alignment, whose wooden superstructure was decayed and dilapidated. The space between the cribs was filled with earth.

An attempt had been made by interested parties to reconstruct the dock by cutting down the old wooden superstructure to low-water level and replacing it with one of concrete. After completing 90 ft. by this plan it became evident to the owner that the substructure of the old cribwork was not strong enough to support the concrete superstructure. This was disclosed by the crushing out of the cribs under the load of concrete. Another objection was that the concrete superstructure would necessarily have to conform to the line of the cribs and so would not correct the irregular alignment of the crib substructure. After a careful examination by the engineer for the new owners it was decided to adopt some scheme by which the new structure should be entirely independent of the old and which would straighten the irregular alignment of the old dock.

With these features predominating, the reconstruction was again commenced by driving 16-ft. anchor piles into the earthfill at a distance of 30 ft. from the proposed alignment of the finished dock. The piles were driven 8 ft. apart and connected at a depth of about 1 ft. beneath the surface of the earthfill by 12 x 12-in. anchor timbers. The lines for the new dock had previously been determined so that the structure would just clear the bottom of the old cribs at the points where they projected the fartherest outward, thus requiring the minimum amount of backfill consistent with straight alignment.



Cross-Section of Timber Crib Pier

The average depth of water adjacent to the dock is 17 ft. and the harbor bottom consists of shale rock overlaid by from 1 to 2 ft. of mud and gravel. A dipper dredge was used to remove the mud and overlying materials from the rock along the proposed dock line. After this had been accomplished, holes 12 in. in diameter were drilled 4 ft. into the rock 8 ft. apart. Two No. 14 Cyclone drills, with chisel-shaped bitts, mounted on a scow, were used for the drilling operations. The scow was held rigid by means of timber spuds. Each hole was drilled to within approximately 1 in. of the proper depth, care being taken to have the bottom of each hole slightly high. Immediately after the withdrawal of the drill a 10-in. 40-lb. I-beam was set into the hole by means of the derrick on the drill scow.

After thirty or forty I-beams had been placed the scow was winded around and the I-beams tapped down to grade by means of a pile-driver hammer. An engineer's level was used to ascertain when the tops of the I-beams reached the grade of 4 ft. above mean lake level. A 1½-in. tie rod was then run from the top of each I-beam to the anchor timbers.

Two 4 x 4-in. angle irons, 12 in. long, were riveted to the outer flanges of each I-beam as brackets. The lower angle or bracket was placed on a line with mean lake level, while the upper one was placed about 2½ ft. higher. The angle irons support upper and lower lines of 12 x 12-in. white oak wale timbers, joined by 2-ft. ship scarfs.

Placing Sheeting

The 6 x 12-in. sheeting was sunk, for the sake of convenience, in single horizontal rows 200 ft. long, these sections being joined by short sections of one panel in length. The first sheeting course was secured to the I-beams by U-bolts and each successive course was edge-bolted to the underlying course with 22-in. round drift bolts. At regular intervals further courses were fastened to the I-beams by U-bolts, and in this manner the sheeting slid down the I-beams by the weight of the courses applied at the water surface until the first course rested on rock bottom. The sheeting was placed by the derrick scow. As soon as the sheeting had been entirely placed the tops of the I-beams were brought into exact alignment by taking or letting out the tie rods. This was done by either tightening or loosening the nuts behind the anchor timbers. As soon as the I-beams were lined up a backfill of earth, slag and cinders was placed.

I-Beams in Place Outside of Old Crib Ready for Sheeting and Tie Rods

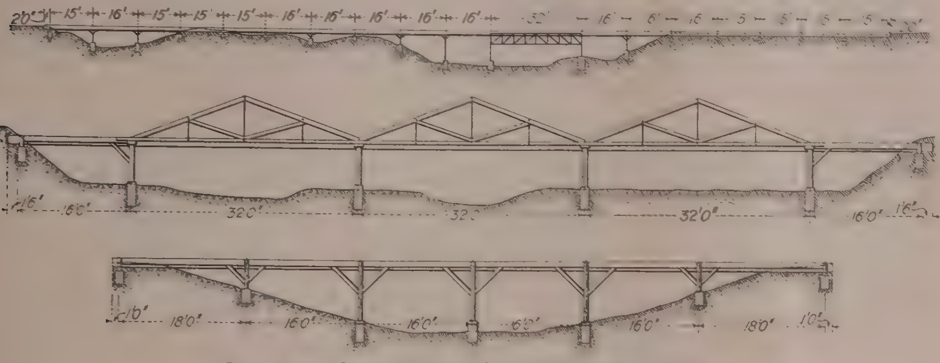
A compound cap of two 8 x 16-in. white oak timbers in 16-ft. lengths was mortised across the tops of the I-beams. The two timbers were fastened together by 18-in. screw bolts and held to the I-beams by U-bolts. All of the timbers above mean lake level were treated with two coats of Carbolineum as a preservative before being placed in the work. The plan outline of the new dock as now constructed is 700 ft. 6 in. in length by 124 ft. 2 in. in width. The new structure is entirely independent of the old cribs and all re-entrant angles in the old alignment have been corrected.

The work was done under contract by the Ashtabula office of the Public Utilities Construction Company, of New York City, and the plans for the dock construction were devised by its president and general manager, Mr. E. C. Bowen, Jr.

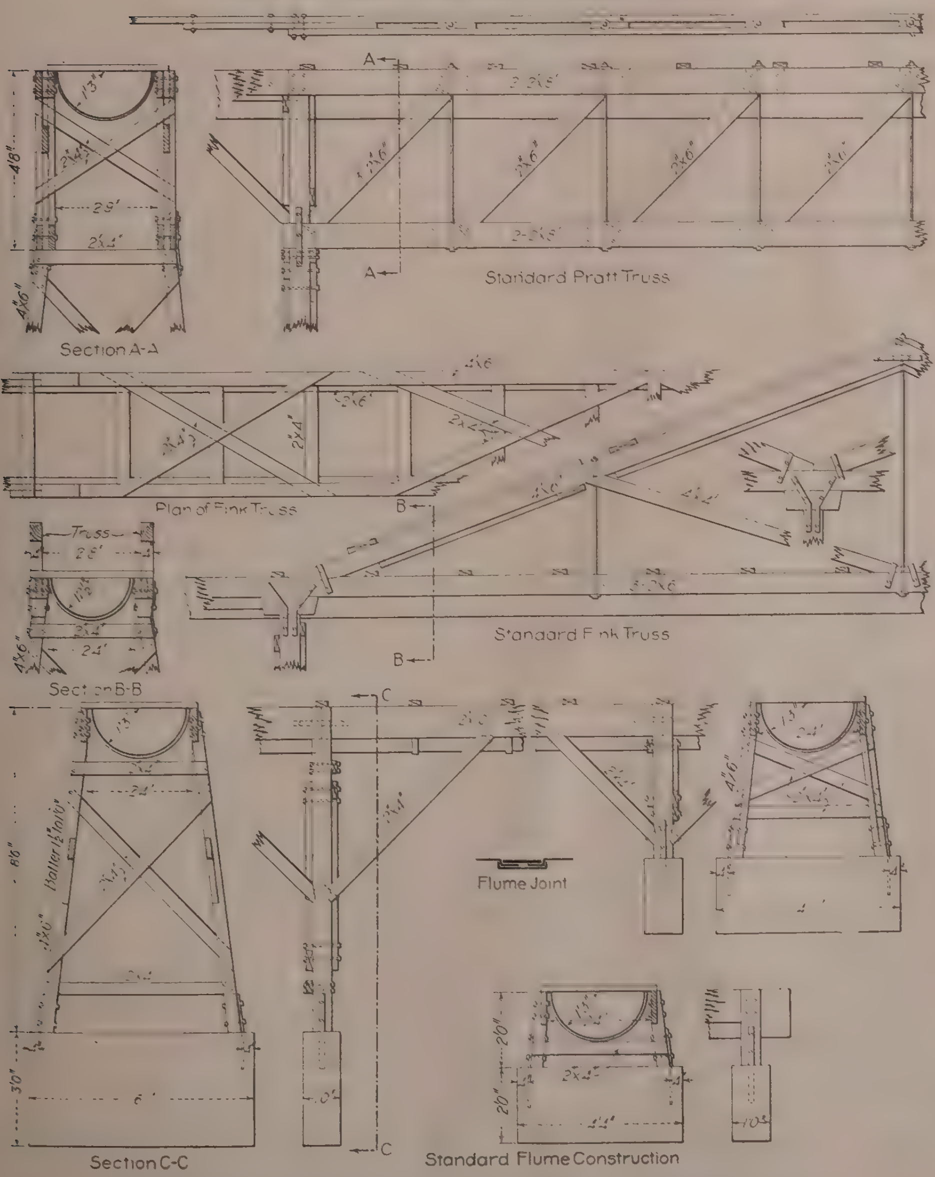
Light-Iron Irrigation Flume

On the Pala Indian Reservation in San Diego County, California, the U. S. Indian Service officials have installed irrigation flumes of light construction. The work is located about 25 miles from the nearest railroad station, which necessitated the use of construction materials of light weight. American ingot-iron smooth flumes were selected and installed on a light wooden substructure. The shapes used are semicircular, varying from 12½ to 15 in. in radius, and were delivered in 2-ft. lengths.

In places where these sections butt against each other the joints are supported by 1-in. channels, curved to fit around the flume. These channels are rounded and threaded at the ends, which are taken through 4x2-in. wooden cross-braces from which the flume is suspended on square nuts and cast iron ogee washers. To prevent the wind from raising the sections and opening up the joints interior butt straps are provided fitting into the offset ends of the flume sections to render the inside smooth. These straps butt at the ends against the under side of the cross-braces. These braces are laid without spiking on two longitudinal 2x10-in. planks running one on either side of the flume and bringing the load of the flume to wooden bents. Should the span exceed 16 ft. Fink or Howe trusses are used to carry the load. The supporting bents are of the A-frame type, with 4x6-in. legs and 2x4-in. bracing. These bents are anchored to concrete foundation blocks by means of 3-16x 1½-in. steel straps, 24 in. long. The width of these blocks is 1 ft., the depth is from 1 to 3 ft. and the length varies with the height of the respective bents.



Spans and General Elevations of Timber Trestles



Details of Timber Structures Supporting Light-Iron Irrigation Flume

It is stated that the iron flumes are especially suited for the conveying of water on irrigation districts, because iron is practically immune to the alkalies prevalent in the soil and waters of semi-arid regions.

Carrying-Capacity Tests

After the flumes were completed tests as to the carrying capacity were made. It was then found that the coefficient of roughness in the Kutter formula was 0.010 for a 30-in diameter flume. A shorter flume gave a slightly lower value, but it was considered less trustworthy on account of the shortness of the flume. The value of $n = 0.010$ corresponds also with the value given by Trautwine for smooth, new iron pipes.

Detail View of Light-Iron Flume

The official records show the cost of the flume with its substructure of painted surfaced selected lumber and concrete piers to be $2.61 per lineal foot; this cost, taking into consideration the character of the country and the long transportation, is considered moderate.

The work was done under the direction of Mr. C. R. Olberg, superintendent of irrigation, U. S. Indian Service, with Mr. H. K. Palmer, assistant engineer U. S. Indian Service, in immediate charge of the field work, and who also conducted the tests. The iron sections of the flume were delivered by the California Corrugated Culvert Company, of West Berkeley, Cal.

Closure of Beulah Levee Crevasse

In the article on the closure of the Beulah levee crevasse on page 4 of the Engineering Record for July 5 the location of Rosedale, the principal town in the affected district, was inadvertently omitted from the general map. This town is on the Yazoo & Mississippi Valley Railroad at the siding north of the steam shovel pit tracks, some distance north of Lake Beulah.

Tests of Reinforced-Concrete Slabs under Concentrated Loading

Abstract of Paper, and Discussion, on the Determination of the Effective Width of Concrete Slabs, Presented before the American Society for Testing Materials

When an engineer is confronted with the design of a reinforced-concrete slab, to be subjected to a concentrated load, his fundamental consideration is the width of slab that may reasonably be counted as effective in carrying the load. It is necessary for him to assume the value of what may be called the effective width of slab when using the ordinary theory of rectangular beam design. To one who has been confronted with the problem of safely and economically making this assumption, tests that will place his judgment upon a scientific basis will be welcomed.

Investigations of this nature have been made by Mr. A. T. Goldbeck, testing engineer of the Office of Public Roads, and the information that he has gathered, while not complete, is worthy of consideration. His paper, presented before the American Society for Testing Materials at the Atlantic City meeting, June 24-28, invoked discussion by Mr. W. A. Slater, first assistant of the Engineering Experiment Station, University of Illinois, where similar investigations have been conducted.

Table 1—Description of Reinforced-Concrete Slabs

Slab No.	Dimensions, ft. Length	Dimensions, ft. Breadth	Depth in. Total	Depth in. Effective	Approximate weight, lb.	Percentage of steel Longitudinal	Percentage of steel Transverse
679	13	6	7	6	6830	0.77	....
705	9	7	5	4	3940	0.91	....
706	9	7	5	4	3940	0.91	0.41
730	9	7	6	5	4720	0.91	....
736	9	7	4	3	3160	0.60	....
737	9	7	7	6	5520	0.75	0.33

Mr. Slater's conclusions concur with those of Mr. Goldbeck. An abstract of the original paper and the discussion in full are presented in this article.

Description of Specimens

All specimens were made of concrete mixed in the proportion 1:2:4. One-half-inch plain round steel rods, having a yield point of 38,800 lb. per square inch and a tensile strength of 73,000 lb. per square inch were used exclusively for reinforcement. Compression cylinders, 8 in. in diameter and 16 in. high, were molded at the same time the slabs were made, and of the same concrete.

The slabs were molded on heavy oiled paper, laid on the floor of the laboratory in dimensions as given in Table 1. The center of the steel was placed 1 in. from the floor, so that the total depth of the slab was 1 in. greater than the effective depth. Deformation measurements were taken both at right angles to the supports and parallel with them, in sufficient numbers to determine the magnitude and distribution of the deformation in each direction.

The deformation measurements were taken in the concrete and steel by means of a strain gage of the type designed by Mr. H. C. Berry. In using this instrument its conical points are simply inserted into the drill holes and the dial reading is taken. For measuring deflections, a special instrument was designed.

Results of Tests

From a consideration of the deformation curves it was noted, as might be expected, that the greatest deformation occurred under the load, and that as the supports were approached the deformations became smaller. The local effect of the load is very perceptible in producing deformation in the slab. Although the deformations at the sides of the slab are small, yet there is a very perceptible deformation, even at a distance of 4½ ft. on each side of the load, or over a total width of 9 ft. This indicates that even at 4½ ft. from the load the slabs investigated exerted considerable resisting moment. In a general way the maximum deformations in the same slab seem to be proportional to the loads applied. The deformations on the top of the slabs parallel to the supports showed a compression very small in magnitude.

An estimate of the width of each slab that might reasonably be considered as carrying the load—assuming a uniform distribution of the safe working stress over this width—was obtained from the curves of deformation. Since the resisting moment of a slab is dependent on the stresses in the top fibers, and these in turn are measured by the unit deformations of these fibers, the resisting moment is proportional to the ordinates to the curves of unit deformation at the center of the slab. If the unit deformation be considered as uniformly distributed with a constant ordinate equal to the maximum ordinate in the actual curves of deformation, the width of the slab, having a resisting moment equal to that of the slab tested, may be obtained by equating the area of the theoretical rectangle of deformation to that of the actual curves of deformation. In this way the effective widths of the slabs given in Table 2 were obtained.

Table 2—Estimates of the Effective Widths of Slabs under Working Stresses

Slab No.	Span, ft.	Effective width, ft. From curves	Effective width, ft. $b = \frac{M}{\frac{1}{2} f_c jkd^2}$	Ratio of effective width (from curves) to span
705	6	5.25	4.66	0.9
730	5	5.5	5.0	1.1
	6	5.75	5.92	0.96
737	5	6.0	6.0	1.2
	6	7.16	5.25	1.2

These values seem to indicate that the effective width of the slab hovers in the vicinity of the span length. This cannot be stated as a general proposition, for doubtless factors other than the span length, such as depth, percentage of reinforcement, etc., have influence on the effective width. The values given in the table are merely approximate estimates and are not exact. To obtain correct values for the effective width, it will be necessary to have the slab of such a width that the deformations vanish at the sides, and in the slabs tested this is not the case. Moreover, many more tests will have to be performed before any definite conclusion can be drawn regarding the influence of all of the variable factors on the stress distribution. It is interesting to note that when the ordinary straight-line formula for resisting moment ($M = f_c jkbd^2 \div 2$) of a rectangular reinforced-concrete beam is used, and the concrete is considered as governing this resisting moment, the effective width of slab obtained, when the modulus of elasticity of the concrete is taken at 2,000,000, checks in most cases with that obtained from the curves. These widths are given in the second column of Table 2 under "Effective Width," and calculated using values for f_c corresponding to the maximum were unit deformation under the load taken from curves which represent very closely the conditions that obtain under working loads.

Deformation and Deflection

Curves plotted from the unit deformations on the bottom of slab No. 679 are of principal interest in showing that it is almost useless to attempt to obtain the stress distribution and effective width from readings in the steel. Great irregularities occur in these curves. Where peaks in the tension curves occur it was discovered that cracks had formed in the concrete within the spaces over which those particular measurements were taken, and indicate a large stretch in the vicinity of the cracks. There is a much smaller stretch on each side of the cracks where the concrete assisted the steel in tension.

The curves of deflection, as well as those of deformation, are somewhat complicated and for this reason are not reproduced here. They are of principal interest in showing the shape assumed by the center line of the slab parallel to the supports when the loads are applied. As in the case of the deformation curves, the deflection curves show the local effect of the load in deforming the slab. The maximum deflections for the different loads on the same slab seem to be approximately proportional to the loads applied, when the loads are not excessively large.

Table 3—Central Loads at Failure

Slab No.	Span, ft.	Bearing area, sq. in.	Central load, lb.
679	11.5	18	21,500
706	3	28.3	42,800
730	6	28.3	24,700
736	6	28.3	7,560
737	6	28.3	34,200

Failure of Slabs

The maximum loads applied to the centers of the slabs at failure are given in Table 3. Slab No. 679, although without any transverse steel whatever, cracked almost in a straight line across the center from side to side, and failed probably by tension in the steel. No. 706, a wide slab tested on a short span, failed, as might be expected, by shear. The bearing cylinder, 6 in. in diameter, punched a hole in the center of this slab at failure, and the concrete was badly cracked under the load. The cracks extended on each side of the load over a width of about 5 ft. No. 730, tested on a 6-ft. span, failed probably by tension in the steel, and, unlike No. 679, two diagonal cracks were formed extending over a width of about 5 ft. No. 736 also failed by tension. It was the thinnest slab tested, and, in addition to the diagonal cracks, others developed at right angles and parallel to the supports. No. 737, like No. 730, and notwithstanding the fact that it was reinforced transversely with 0.33 per cent of steel, developed diagonal cracks, extending in this case over a width of about 6 ft. The width over which these tension cracks extend in all of the slabs tested is very instructive and is indicative of the width over which the bending movement was most largely resisted at failure. Perhaps it is significant that in the case of Nos. 706, 730 and 737 the widths over which the cracks extend correspond reasonably well with the calculated effective widths under working loads.

Conclusions

No definite conclusions can be safely drawn from the tests thus far made, but the data collected are useful in indicating that in slabs of the kind described and of 3 to 6-ft. span length the concentrated central load might quite safely be considered as carried by a width of slab equal to not more than eight-tenths of the span length.

Discussion by Mr. W. A. Slater, University of Illinois

The tests on which this discussion is based were made on a large number of beams having a span of 4 ft. and widths varying from one-half the span up to twice the span, on one beam having a span of 10 ft. and a width half as great, and on two beams having a span of 30 in. and a width 3.2 times as great. In so far as these tests may at present be considered to give basis for conclusions they agree closely in quality with those reached by Mr. Goldbeck, though there are some quantitative variations from his results.

Mr. Goldbeck has stated that his deformation measurements were taken generally upon the concrete, and that measurements upon the steel give such erratic results as to be of little value. The writer agrees that measurements on steel are likely to be erratic, but believes that, since the steel stress is more likely to be critical than the concrete stress, conclusions should be based on the results of measurement of steel stress. This seems especially important in view of the form of the typical steel stress-distribution curve shown in Fig. 1. Here it appears that the variations between the stress at the center of the beam and that at the edge is much more extreme than is found in the concrete stress. The effective width then (that width in which if the entire moment were resisted a uniform stress would be developed which would be equal to that actually found at the center of the beam) should be less than that determined on the basis of concrete stresses, and deductions as to relation between span and effective width will be more conservative.

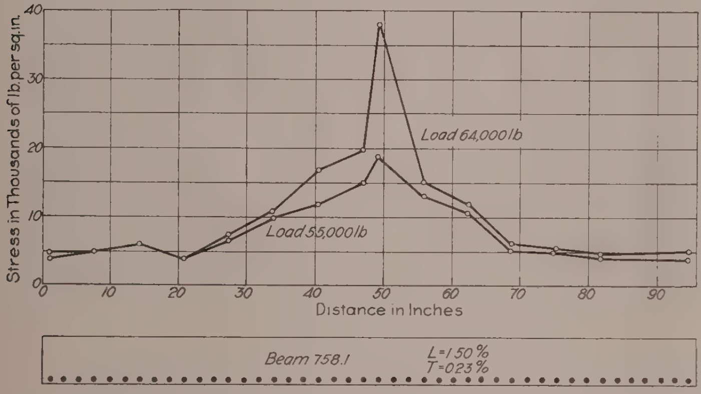


Fig. 1—Typical Steel Stress-Distribution Curves

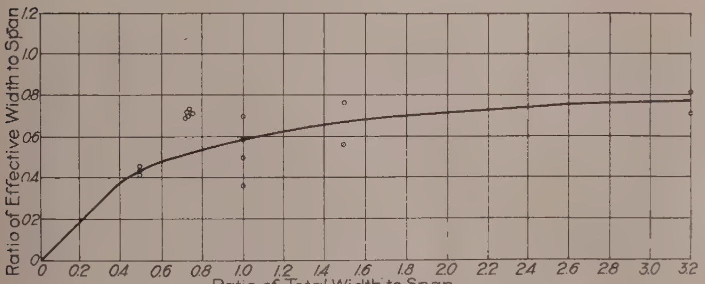


Fig. 2—Relation of Width Ratios to Span

In Mr. Goldbeck's paper it is stated that in the test of slab No. 679 cast-iron plugs drilled for measurement were wired to the reinforcing. Wherever a crack opens there must be local slipping of the bar through the concrete, and since these cast-iron plugs must move with the concrete rather than with the steel any movement of the plugs will indicate essentially the opening of cracks rather than the stress in the steel. If, then, measurements for steel deformation were taken on these plugs, the results should be expected to be erratic. More consistent results have been obtained from measurement directly upon the steel.

Ratio of Width to Span

Fig. 2 shows the ratio of the effective width to the span as determined from the measured steel stresses in the University of Illinois tests. It is to be noted that there is apparently a sudden change in the distributing action between the points where the width is three-fourths of the span and that equal to the span respectively. The curve was made to conform to the lower value. Apparently at a width of about twice the span this ratio becomes constant, or nearly so, and equal to about seven-tenths of the span for the mean curve. The fact that Mr. Goldbeck's values of the ratio of effective width to span are higher than those found by the writer may be due partly to the fact that they are based on concrete stress instead of steel stress and partly to the fact that the load in his tests was applied at the one-third points of the span.

That the method of loading may be expected to influence the effective width may be seen by comparison of Fig. 3 of this discussion with the failure of slab No. 679 as described in Mr. Goldbeck's paper. It has been pointed out that the distance between the radiating cracks where they reach the edge of the beam is to some extent indicative of the effective width, and it has been suggested that for a very wide beam this distance is about equal to the span. In other words, the angle ϕ between the center line of the beam and the general direction of the crack is constant and its tangent equal to 1.0. If the distance from the load to the support be called x, the effective width b, the span s, and the width of loading beams e, then

$$b = 2x \tan\phi + e = 2x + c.$$

If $x = s/2$, $b = s + e$. If $x = s/3$, $b = (2/3)\,s + e$. The former value agrees approximately with Mr. Goldbeck's results and the latter with those obtained by the writer. If as a conservative coefficient $\tan\phi$ be taken as 0.8, as is proposed by Mr. Goldbeck, b becomes $0.8s + e$ and $0.53s + e$ for beams loaded at center and at one-third points respectively.

From Table 4 it is seen that when the width assumed in the calculation is the total width of beam the calculated stress agrees reasonably well with the average measured stress. When the width assumed is the effective width the calculated stress corresponds with the maximum measured stress. Thus, if the effective width at any load be known a beam may be designed so that the maximum stress will be approximately equal to any stress which may be assumed in design. However, it must be borne in mind that the determination of the effective width has been based on the stresses observed at loads less than the ultimate. While it may be expected that in a beam designed by the methods outlined here the stress in the center of the beam at the design load will not be larger than the assumed working stress, the factor of safety may be less than that which is indicated by the ratio of the yield-point stress to the working stress assumed.

Relation of Reinforcement

If from the maximum loads given in Table 3 of Mr. Goldbeck's paper the bending moment be calculated and equated to the expression for resisting moment as given in the equation $M = pf_s\,jbd^2$, it is possible to solve for b the width of beam which must be effective to produce the yield-point steel stress of 38,800 lb. per square inch, the value for the reinforcing bars used by Mr. Goldbeck. These calculations give values of the effective width sometimes higher and sometimes lower than that obtained by the method employed in Table 2. In the tests made at the University of Illinois it seems in general that the effective width as based on the ultimate load is less than that obtained on the basis of steel stress at loads less than the ultimate. The meaning of this may be either that the steel stress increases faster proportionally than does the load or that the failure is due to something other than stress in the steel—perhaps to transverse tension in the concrete.

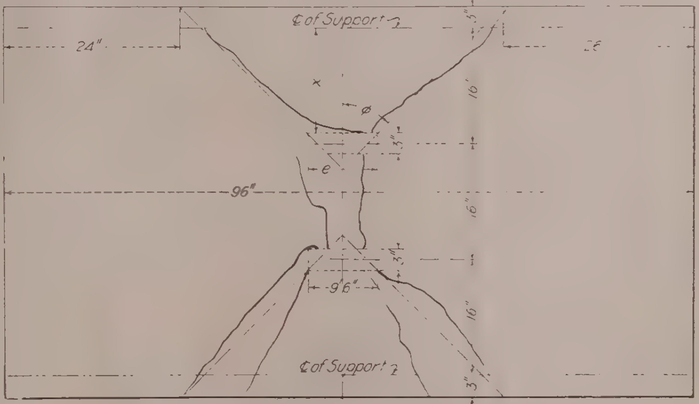


Fig. 3—Radiating Lines of Failure in Test Slab

An attempt has been made to fix a criterion to determine whether or not the assumption of an effective width equal to $0.8x + e$ is applicable to any given case or not; but so far nothing general has been determined. In view of the uncertainty which surrounds the

Table 4—Comparison of Measured and Calculated Stresses

Beam No.	Width, in.	Longitudinal reinforcement, per cent	Load considered, lb.	Measured stress, lb. per sq. in.		Calculated stress, lb. per sq. in.	
				Maximum	Average	Based on effective width	Based on total width
765.1	72	1.04	32,000	20,000	10,300	21.500	11,100
765.2	72	1.04	40,500	29,000	11,000	37.000	14,100
753.1	48	1.51	42,500	21,000	12,300	27.000	15,900
753.2	48	1.51	42,000	22,500	15,600	22,500	15,600
763.1	48	1.04	31,600	42 000	15,600	46.000	16,500
763.2	48	1.09	40.300	42.000	21,400	40.000	20,200
762.1	36	1.02	38,500	26,500	25.300	28,300	27.300
751.1	24	1.07	37,000	35 000	29,600	44,500	37,700
752.1	24	1.07	41,500	37,000	33,600	32.000	29,800

matter it seems to the writer that it would be in the interest of safety to use an effective width, $(2/3)\,x + e$, and to limit the longitudinal reinforcement to 1 per cent or less. In view of the statement which follows, indicating that the effective width is independent of the amount of longitudinal reinforcement, it may appear inconsistent to limit the latter to 1 per cent. This limitation is placed because of the possibility that in a beam with a large amount of longitudinal reinforcement and a relatively small depth failure may be caused by transverse tension in the concrete and not by longitudinal steel stress. Under such circumstances the effective width would not be a significant factor in the calculation of the ultimate strength of the beam.

It must be understood that the method proposed for determining the effective width represents the conditions which may be reached in the limiting case when the span is finite and the width infinite. For widths greater than $2s$ it probably is a sufficiently close approximation to be used without fear of serious error. For a beam whose width is less than twice the span the proportionate reduction in the effective width is indicated in Fig. 2.

INFLUENCING FACTORS

In Mr. Goldbeck's paper it is mentioned that "factors other than span length, such as depth, percentage of reinforcement, etc., may have influence on the effective width." Experimental evidence has been found indicating that for loads below the ultimate the effective width is independent of the per cent of longitudinal reinforcement. By analogy it would be expected to be independent of depth also. In Table 2 of Mr. Goldbeck's paper, however, a slight tendency is apparent for the effective width to increase with the depth. On the other hand, tests at the University of Illinois indicate a slight tendency for the effective width to vary inversely with the depth. In view of the fact that the variation in effective width corresponding with variation in depth is very slight, and in opposite directions in the two cases, it would seem that depth as a factor influencing the effective width is unimportant. Thus, it is indicated that while factors other than span length may have influence on the effective width, these other factors are relatively unimportant. Therefore for beams without transverse reinforcement the method observed in the paper and in this discussion of considering the span length as the important factor governing effective width seems to be justified.

In the oral presentation of his paper Mr. Goldbeck has called attention to the fact that assuming the effective width as a constant proportion of the span results in the obtaining of a constant depth for all beams required to support the same concentrated load P regardless of the span. This does not seem unreasonable when it is remembered that for the same total load similarly placed on the span the moment and the effective width both increase directly with the span, thus keeping the moment per unit of width constant. This is shown by the following equations:

Let $s_1 =$ span, $b_1 =$ effective width, and $M_1 =$ moment on span s_1. Then $M_1/b_1 =$ moment per unit of width.

For any other span, ks_1, $kM_1 =$ moment and $kb_1 =$ effective width.

The moment per unit of width now becomes $kM_1/kb_1 = M_1/b_1$ as before.

The moment considered here, of course, is only the live-load moment. The dead-load moment shows a substantial increase with the span, and the corresponding increase in depth of slab or per cent of reinforcement is considerable.

In this discussion all that has been said applies only to beams without transverse reinforcement. Tests have been made on beams having various percentages of transverse reinforcement, and so far as deductions are justified these tests have not shown conclusively that transverse reinforcement for the purpose of resisting transverse bending stress is economical. As a general consideration, however, it seems that there ought to be a small amount of transverse reinforcement for the purpose of distributing deformation due to variations in temperature.

Dominion Land Surveys

By J. A. Macdonald, Ottawa, Ontario

To be a Dominion land surveyor is no simple matter. One must know mathematics from the ground up, and one must serve three years, six months of which shall be in the field, with a surveyor or in the surveyor-general's office. A man may become a civil, electrical, mechanical or mining engineer without passing a final rigorous examination, but to become a Dominion land surveyor a final examination in mathematical theory and practice must be passed at the end of the three years. After passing this examination, however, one is ready for business.

After completing a technical college course and attaining a degree one must, if he desires to become a "D. L. S.," take the preliminary examination, which involves plane and solid geometry, spherical trigonometry, mensuration, etc. The final examination, which occupies six days and may be taken three years later, embraces the same subjects, and also measurements of areas and subdivision of lands, description of deeds, astronomy and surveying.

Astronomy is the most difficult subject. Azimuth, latitude and time must be observed by various methods. For example, the latitude of a place must be determined from an observation of the meridian altitude of the sun or a star, and also from an altitude of a star on the prime vertical; local time and azimuth must be found from an observed altitude of the sun or star, and calculation of the direction of the meridian must be made from an observation of a circumpolar star at its greatest elongation, and at any hour-angle. The ability to do practical work with the transit is also a requirement, as well as the ability to make a fairly neat and correct plat.

COMPENSATION

Surveyors working by the day are paid $9 per day, including Sundays and holidays, from the time of leaving their homes. They are allowed an ample amount extra for living expenses. Those working under contract are allowed $5.50 per mile of line surveyed. A further payment at the rate of $40 per mile for anything up to 10 chains to a section side, and $20 thereafter, is made for opening, cutting and blazing the line through woods, windfalls, underbrush or heavy scrub.

Further payments are made of $3 per mile for section lines surveyed over rough, hilly country and $4 per mile for surveying the meridian outlines of a township where such outlines are included in a subdivision survey contract. A payment of 40 cents per pit is made, and as there are four pits made at each section and quartersection corner, this amounts to $1.60 for each corner.

Traverses of lakes and rivers and connecting traverses are paid for at the rate of $11 per mile. The iron posts planted at each quarter-section or section corner are supplied free. These posts, with the pits and mounds, form the corner monuments.

The cost of surveying a township varies with the area and density of woods it contains. A contractor now averages from $1,500 to $2,000 for the subdivision of a township. He must, of course, find his help and stand all costs and expenses of the survey. Some surveyors under contract make as much as $30,000 gross in a season, running at times two gangs of surveying parties, with two assistants or transitmen.

The number of men needed by a surveyor in the field depends upon the size of his contract. If he has a large contract he will require two transitmen, or "assistants," as they are called, and from twenty to thirty men, besides a cook and one or more teamsters; also five or six horses and wagons, tents, cooking utensils and a full supply of provisions.

Assistants to be eligible must have passed the preliminary examination previously mentioned. They are paid $4 per day and expenses.

Upon the receipt in the surveyor-general's office of a preliminary report on the completion of the subdivision or other work of a township, 75 per cent of the amount coming to the surveyor is paid him. The remaining 25 per cent is not paid until the work is inspected and shown to be done satisfactorily and the full notes and returns of the survey are completed and sent, as required, to the head office.

A surveyor before being able to obtain a contract must give a bond to the Minister of the Interior, jointly with two sureties, or he may purchase a bond from an approved guarantee company.

METHODS EMPLOYED

In making block surveys the base lines are first run. These base lines are 6 miles apart, north and south. The first base line is the international boundary. These base-line surveys require careful work, and only the best men are delegated therefor. The northern and southern base lines are parts of a polygon described on a parallel of latitude by laying off chords thereto, the successive township sides forming the northern and southern outlines of the block.

All these lines are measured twice. This is effected by having two sets of chainmen using a continuous steel band chain. The leading chain is the length of a standard Gunter's chain; the following one, used as a control, is 500 ft. in length. Corrections for temperature are made frequently. A clinometer is used for chaining over rough ground.

The instrument used for the survey of block outlines is a 6-in. reiterating theodolite of which the horizontal circle is read by two microscopes. A 4-in. vertical circle read by a vernier serves as a finder for stars in the daytime. The telescope has an objective of 1⅞-in. diameter and a 14-in. focus. A small magnetic needle attached to the instrument is used to assist in finding stars in the daytime.

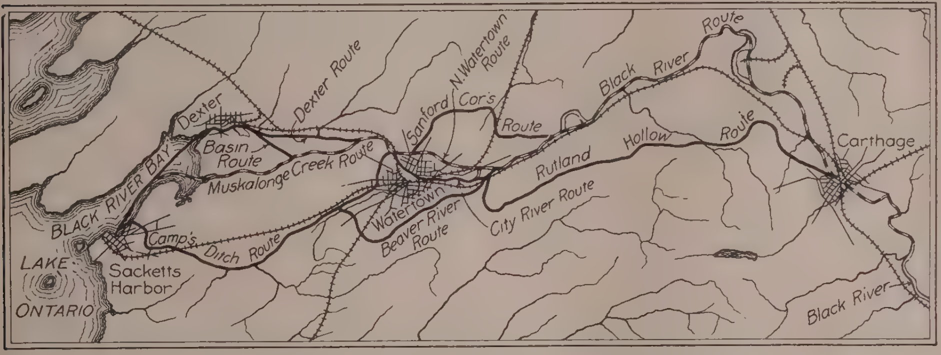
For astronomical work the surveyor must be provided with a sidereal pocket chronometer. The transit is always provided with a striding level for block-survey work, but not for subdivision or contract work.

Base-line surveyors are paid by the day. In producing the line the surveyor employs but one flagman, a forward picket man; a back flagman is not necessary, as the surveyor when about to leave an instrument station can set a picket there himself.

Surveys and Estimates for Proposed Black River Canal

The city of Watertown, N. Y., with a population of about 30,000, is about 8 miles above the mouth of the Black River proper; about 20 miles above Watertown is Carthage, with about 4000 inhabitants. The valley is dotted with industrial works, but the river is navigable only to Dexter. As early as 1838 there was agitation to build a canal from the navigable waters to Carthage. In 1911 the Legislature voted an appropriation of $15,000 to make surveys, plans and estimates for such a canal, and Mr. Louis A. Burns, resident engineer, was assigned to this work. The following notes are taken from his report, which is included in the 1912 report of the state engineer and surveyor.

Two days were spent on reconnaissance. Six days later the survey work was begun; it had been decided to use stadia measurement throughout, and the field work was finished within five months, 175 miles of stadia base line having been run, from which 115 miles of canal were estimated. The office work was completed in less then four months more, the total cost of the investigation being $10,785.62.



Location of the Principal Routes Investigated

Surveys and Estimates

As already stated, all measurements were taken by stadia. Traverses were closed as far as possible, and azimuths were frequently checked by observations on Polaris. Accurate wye levels were used to establish frequent benchmarks to check the stadia work, the levels being run from existing U. S. Geological Survey benches.

Topographic maps on a 100-ft. scale were plotted from the stadia notes, numerous existing topographic maps being utilized as far as possible. The maps were matched together on a large floor and the center line of the canal was projected on them. Prism and embankment areas were taken from tables previously computed for the purpose, and to simplify computations rock was assumed to stop with a full rock section at a certain station, and a full earth section was assumed to begin at the same point.

As the estimates were based upon the depth of canal and size of lock used in the Barge Canal work, it was possible to secure costs on Barge Canal locks finished or under way with lifts varying from 10 to 40 ft. From these figures a curve was plotted, from which costs for the lifts required were picked off. Another curve was plotted for an alternative estimate for locks 140 ft. shorter, the "full-sized" locks being 328 x 28 ft., 12 ft. deep over miter sills. Barge Canal data were utilized where available in estimating the costs of bridges and dams.

The following unit prices were used in the estimates, all of the rock being Trenton limestone:

Rock excavation, dry	$1.50
Rock excavation, subaqueous, with possibility of unwatering	2.50
Rock excavation, subaqueous, with no probability of unwatering	3.00
Earth excavation, soft silt, dredge work	.20
Earth excavation, stiff, steam-shovel work	.35
Concrete	6.50

After conclusions had been reached as to the desirable route a careful study of the water supply was made. On the basis of utilizing the river from Carthage to Watertown it was found that more than twice as much water should be supplied at Watertown as at Carthage. Gages are being read on three streams from which water may be taken, on two of which existing dams are used, the construction of a weir having been necessary in the third. The report contains information about numerous other streams, possible sites for dams, and the probable amount of water available.

Forty-four long and short lines were estimated, from which ten combinations are possible. The principal ones, however, are those shown on the map. From Carthage to Watertown the route following the river is recommended in spite of its higher cost, for the reasons that it would serve the shippers along the river and would minimize the amount of water to be supplied.

Alternative Routes

Of the four routes through or around Watertown, the Sanford Corners and the Beaver Meadows lines are rejected because they are long and costly and do not reach many of the shippers. The City River route would reach the shippers, but would be costly and hard to build and operate. The North Watertown line is, therefore, recommended, as it is the cheapest and is nearer to the shippers than any except the City River route.

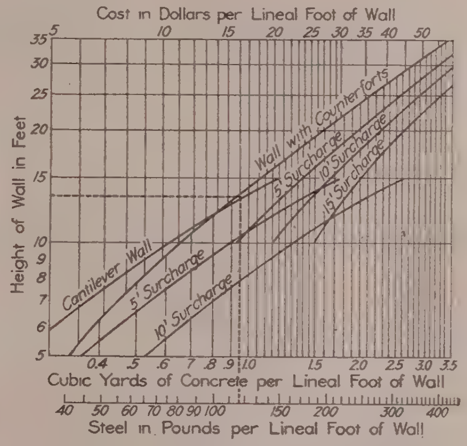
Of the three main routes from Watertown to navigable water the Muskalonge Creek route is the one recommended. The Camp's Ditch line shows to disadvantage as to cost, length and service to the shippers. The Dexter route would serve the shippers, but would cost more than the one chosen, would be beset by precarious construction conditions, and would require the maintenance of a channel more than a mile long through a deep alluvial deposit at the mouth of the river. The Basin route would avoid the worst construction difficulties, but would not escape the bar at the river's mouth, and would cost more than the Muskalonge Creek line. The latter would pass within a short distance of a number of mills, and boats of 4-ft. draft could easily bring cargoes from Dexter to the harbor which would be established in Muskalonge Bay. This harbor could be built very cheaply, and is another point in favor of the route recommended.

The estimated cost of the improvement with locks 328 x 28 ft. is $16,300,000, 20 per cent having been added to the computed figures for contingencies. With the smaller locks the total figure would be reduced to $14,600,000.

Diagram of Cost of Retaining Walls

By E. F. Kriegsman, Assistant Engineer, City Engineer's Office, San Francisco

The diagram shown in this column is one that the writer has had made in order to ascertain quickly the approximate cost of retaining walls. It is useful in determining approximate cost of walls in connection with regrade work, viaduct and tunnel approaches, etc. Usually one has to wait for the designer to ascertain the quantities before a preliminary cost report can be made. This manifestly has its disadvantages.



Approximate Cost of Retaining Walls

In constructing the diagram about forty wall designs were investigated and the quantities computed. The results were next plotted on logarithmic cross-section paper and average curves drawn representing the amounts of concrete and steel reinforcement for various heights of wall having a 1-ft. length. An average price of $17 per cubic yard for concrete, steel, forms and finish was applied to these yardages. All that one needs is the grades of the top and bottom of the proposed wall, when one can readily approximate the cost of the finished wall with the use of the diagram.

The price of $17 per cubic yard may not meet the requirements of many engineers, as local conditions vary considerably. If another unit price is desired all that is necessary is to trace the cost scale on tracing cloth and move it horizontally until the desired cost is in the same vertical with the 1-cu. yd. line in the diagram now occupied by $17 at the top. The new location of the cost scale will give the desired cost for the given local conditions.

For illustration the dotted line shows that a cantilever wall 14 ft. high, without surcharge load, would contain 0.95 cu. yd. of concrete and 116 lb. of steel reinforcement and would cost approximately $16 per linear foot. Excavation must be figured separately.

This diagram has proved quite useful to our estimators and the writer hopes that others may find it of assistance.

Auto-Bus Lines, or stage coaches, operating over improved New York state highways are required by law to take out certificates of convenience or necessity before they are allowed to maintain a regular schedule.

Value of the Classics in Modern Education as Viewed by an Engineer

An Address Delivered before the Classical Association of the Middle West and South by Charles P. Steinmetz, A.M., Ph.D., Chief Consulting Engineer, General Electric Company

The study of the classic languages, Greek and Latin, including the literature of these languages, and the history of the ancient times, was, until a few generations ago, the foundation of all education since the first awakening of the German races in the early Middle Ages.

During the middle of the last century the tendency made itself felt of reducing and finally entirely eliminating the classics from modern education in favor of "more utilitarian" subjects—science, modern languages, etc. The flood tide of this tendency has just passed, and it is beginning to be realized that the result of the narrow utilitarian training has been a failure. Very few of the professional and business men with this training have reached prominence in scientific and national life, and the urgent need of a return to a broader education is becoming more realized from year to year.

Ours is an age of science and engineering, of industrial development and progress. The unfettering of the forces of human initiative and ability by the French Revolution at the end of the eighteenth century, the opening up of the vast resources of our continent, gave opportunities never existing before, and impatiently the youth chafed against wasting time in education, instead of "doing things" by grasping the opportunities. Fortunately for the intellectual progress of the race, these opportunities are gone, and intelligence and knowledge again are replacing chance and grasping. That means that education becomes the essential requirement in determining success in life.

Education Defined

Education is not the learning of a trade or profession, but the development of the intellect and the broadening of the mind afforded by a general knowledge of all the subjects of interest to the human race. These enable a man to attack intelligently and solve problems in which there is no previous detailed experience guides, and to decide the questions arising in his intellectual, social, and industrial life by impartially weighing the different factors and judging of their relative importance. These problems, and thus the educational preparation required to cope with them, are practically the same in all walks of life, and the general education required by the engineer, the lawyer and physician is thus essentially the same. The only legitimate differences in the preparation for life's work required by the different professions are those pertaining to the specific instruction and study of the details of the particular branch of human knowledge by which the student desires to make his living.

The amount of human knowledge has grown so vast that no single mind can ever master it all. That means that we must limit ourselves to a part, usually even a small part, of human knowledge—must specialize; and ours, therefore, has been called an age of specialists. It must be realized, however, that the value of the specialist in the social organism is in direct proportion to the general knowledge which he possesses. Special knowledge, no matter how extensive and intensive, is of very little value if not intelligently directed and applied. This requires broadness of view and common sense, which only a broad and general education can give, but which no special training supplies; such special training rather tends to narrow the view and incapacitate the man from taking his proper position as a useful member of society. Examples of this we can see all around us, especially in the business man, the lawyer and, more still, in the engineer, because the vocation of engineer is especially liable to make a man onesided. By dealing exclusively with empirical science and its applications the engineer is led to forget or never to realize, that there are other branches of human thought besides empirical science, and equally important as factors of a broad general education and intellectual development. An introduction to these other fields is best and most quickly secured by the study of the classics, which open to the student worlds entirely different from the present—the world of art and literature, of Hellas, and the world of organization, administration and of citizenship, of Rome—and so broadens his horizon and shows relative values in their proper proportion and not distorted by the trend of thought of the times.

Pieceworkers and Specialists

There have always been educated and uneducated, skilled and unskilled workers. But with the development of modern industrialism a third class has arisen between the skilled and the unskilled, the educated and the uneducated —men trained to do one thing only, but do this very well and efficiently. We call them pieceworkers when working for wages in the factory; specialists, when receiving salaries as professional men. They are tools, useful when directed by somebody's intelligence, but useless to themselves and to the world otherwise. The product of many of our engineering schools, business colleges, etc., is of this character. Some of these men may become intelligent and educated human beings and useful members of society afterward, it is true, by life's educational efforts, but their schooling did not make them such.

A skilled mechanic may finally specialize in one class of work, but that does not make him an unskilled pieceworker. An engineer, physician or other professional man may devote his time to one branch of his profession; but as long as he keeps up his interest and his familiarity with his entire profession, and with all the problems of the work surrounding him, he has not yet deteriorated into a specialist.

The greatest problem before the educational world to-day is the method of broadening education to counteract the narrowing tendency of modern life and modern industrialism, to produce that intellectual development and broadening of the mind which produces not merely intellectual machines, but citizens capable of taking their proper place in the industrial and social life of the nation—men who can be trusted to direct the destinies of the republic during the stormy times of industrial and social reorganization which are before us.

Modern society is dominated by industrialism, the outgrowth of applied science—that is, by engineering. The entire world has been unified, and whether we travel through the European countries or see the new civilizations of the Far East we find no material differences from the intellectual and social conceptions of our country. Thus the broadening effect of the study of other nations and countries has largely vanished. Wherever we go we meet similar conditions, the same scientific and religious beliefs, the same organization of society, and we are very liable to draw the conclusion that our conditions, our beliefs, our form of society are the best and the only feasible ones, that civilization could not exist without them, and that any radical change would be destructive to civilization. But self-satisfaction means stagnation, and stagnation means decay, and herein lies the foremost danger of our civilization.

Remedy for the Leveling Tendency of Present-Day Civilization

The remedy therefor is the knowledge and familiarity with another civilization, different from ours in character, superior in some respects, inferior in others.

Nobody familiar with Hellas in its prime can ever believe that the highest development of art, science and literature which the world has seen cannot exist in the freest form of democracy—a democracy as free and unrestrained as to be almost anarchism. Nobody familiar with the Alexandrian period can deny that science can flourish under an autocratic monarchy. A purely communistic nation held Hellas for centuries. For centuries the centralized federal government of Rome maintained the peace and guarded the civilization of the entire civilized world, and many countries under Rome's dominion enjoyed a civilization which they never reached before or after.

It is this difference of the ancient civilizations from the present which makes the study of the classics of importance and almost of necessity to-day more than ever before in order to counteract the equalizing and leveling tendency on the human mind exerted by present-day conditions and to give that broadening which is the most important object of education.

Lack of Educational Value in Modern Languages

The modern languages are not in the same class with the classic languages, as they open to the student no new world, no field of thought appreciably different from his own, and I therefore consider them as of little educational value. They have some utilitarian value for the small number of people who travel into foreign countries, and also enable us to use foreign scientific, engineering and other publications which have not been translated. However, their study is so much easier and more expeditious by conversational methods outside or before the higher schools that they hardly find a proper place in the curriculum of higher educational institutions.

The study of the classics involves that of the language, the literature, and the history of ancient times. It has been claimed that practically the same benefit can be derived by studying the classic literature from good translations. This is not so. It is impossible to get into the spirit, the atmosphere of a time and its literature, and to gain a true understanding of its history without knowing the language. It may not be necessary or feasible in many cases to become so familiar with the language as to be able to read fluently all its literature, and translations may then be of service. It must be realized that a translation means something very different to one who is familiar with the history, and somewhat familiar with the language, than it does to one to whom language and history are entirely strange.

There also is considerable utilitarian value in the classic languages, as the terminology of science is entirely based on Latin forms with Greek and Latin roots, and while the student may memorize the terms of his profession, it is difficult, if not impracticable, to memorize all the terms of science with which an educated man must be familiar. This, however, becomes easy to the student of the classic languages, to whom these terms have a meaning. To eliminate the scientific terms is obviously impossible, as the common or English names usually are different in different localities, where they exist at all, and are therefore, indefinite. However, this I rather consider as of secondary moment.

Selecting Classic Literature

It appears to me that there is great need in reintroducing the classics to emphasize the human side, which would bring the classic ideas into relation with modern life. Often the methods of teaching are not the most efficient, and the classic literature is not selected so as to be of the greatest value in broadening the student's view, or to attract and retain his interest as much as possible.

Thus, in Latin the story of war and conquest—of the victory of military organization over mere bravery—in Cæsar's "De Bello Gallico" is interesting and instructive, while the civil war is of less interest. Even to-day Cicero's "De Officiis" is well worth reading.

In Latin poetical selections from Ovid's "Metamorphoses" are easy reading, are a valuable introduction to the classic meter, and are interesting in the parallelism between the myths of the classic world and those of other races (the flood, etc.); but it is hard to understand the retention of the uninteresting plagiarism of the courtier Virgil in the curriculum, while the most important in educational value and interest, Horace, is often neglected. Of all Roman writers Horace probably exerts the most broadening influence on the intellect when read under an intelligent instructor; the change from the distortion in which the relative values of persons and things appear to their contemporaries to the proper proportion in the perspective of history probably is nowhere so clearly demonstrated as in the relation between the "libertino patre natus" and his "protector" and "patron" Mæcenas, whose name has escaped oblivion merely by his favorite's favor.

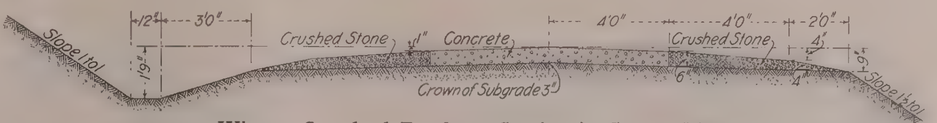
In Greek prose, Xenophon's "Anabasis" is interesting and instructive in many respects, and may well be followed with maps of the country traversed by the ten thousand. Selections from Lucian possibly are the nearest approach to Horace. The Greek drama probably is beyond the scope which can be attempted in a general college course. At least a part of the New Testament may be read in the original, and the value of such reading can hardly be overestimated in showing the meaningless nature of theological controversies on words of imperfect translation. The greatest work of the literature of Hellas, however, is that of Homer; and here again in many American schools the "Iliad" only is read, while the far more interesting "Odyssey" is slighted. The latter, with its tales of travel and adventure, with giants and monsters, should especially appeal to the American boy, and is of far greater educational value than the "Iliad" in its minute description of everyday life at the early dawn of human history and in its word pictures of times and occupations, of the time of the day and of the coming of night and the dawn.

Winona County Concrete Roads

Sixteen Miles of 8-Foot Concrete Roads with Macadam Shoulders

The construction of 16 miles of 8-ft. concrete roads with macadam shoulders in Winona County, Minnesota, presents an unusual example of the successful co-operation of individuals and private capital against local official opposition and limited public resources.

The Minnesota Legislature of 1911 passed a bill, known as the Elwell law, that provides for the building of state highways by counties when the latter are petitioned by six taxpayers who will be liable for special assessment against the improvement. Under this law the state pays one-half the cost, the county one-quarter and the benefited property owners the remaining one-quarter. It also provides that counties may issue bonds for road improvement to run not longer than ten years, and the state, county and benefited property owners are to pay each year one-tenth of their share of the expense of the improvement covered by the bond issue. The property owner must pay the interest on the deferred payments of his share and the county must take care of both the state and county interest payments.



Winona Standard Roadway Section in Cut or Fill

At the time the Elwell bill was passed the constitutionality of the property-owner assessment feature was a matter of conjecture. The Supreme Court a few years before had ruled that rural highways were strictly a public improvement and not for individual benefit and that taxpayers were not subject to special assessment for their improvement. In order to overcome the possible complications arising from this ruling the law was amended to provide that in lieu of assessments contributions might be provided for.

Planning the Winona Road

Following the establishment of this law certain progressive residents of Winona County drew up a plan to build 7 miles of macadam road. Deciding to play safe, the 25 per cent of the total cost of the proposed improvement required from the property owners was raised by private subscription. The interest of the Board of County Commissioners was aroused and its pledge to put the improvement through was secured. Then three members of the board changed their minds. They could not see where there was "anything in it for them" in their districts. This contention was met by a proposal to build five additional miles in another direction. The proposal was satisfactory to them and the money for the additional improvement was raised as before. Then again three of the five commissioners refused their consent and the plan was left in abeyance.

In order to win over a majority of the commissioners to the plan it was decided to build an additional four miles of road in another commissioner's district. The 25 per cent in this case amounted to about $7000. The raising of this amount was difficult. The men of the city of Winona had already contributed nearly $23,000. In this dilemma an appeal was made to the farmers who would be most benefited by the additional four miles.

Farmers Contribute

The work was gone about in a systematic manner. The farmers whose properties adjoined the road were asked to contribute $1.25 per acre; those at some little distance from it —as far as 3 miles—$1 per acre; and those at a greater distance, who would use the road only occasionally, were asked to contribute 50 cents per acre. Contrary to the expectations of some, no money was ever raised more easily or given more cheerfully. A mile and a half of macadam road had, in previous years, been built for the city along this road, the East Burns Valley road, and the advantages and comfort derived probably account for the willingness of the farmers to increase the good-road mileage and pay for it out of their own pockets.

Bids were asked on a 16-ft. road of limestone-macadam construction 7 in. deep, with a 2-in. gravel surface. They were duly re-

Winona County Concrete Road with Crushed-Stone Shoulders ceived and rejected by the county board. The prices were too high. However, the state highway commission had in the meantime been investigating concrete roads to a limited extent. They suggested an 8-ft. concrete road with 6-ft. shoulders of crushed limestone. The suggestion struck the local promoters as a good one; some money was raised, and the members of the county board were taken in a body to Wayne County, Michigan, and to Chicago. They came home enthused and unanimously in favor of the adoption of concrete. New specifications were prepared and one contractor bid $1000 a mile less for the concrete road than his previous bid for macadam.

Adopt Concrete Roads

It was thought better, owing to the limited funds, to reduce the cost per mile by limiting the concrete to a width of 8 ft. The slab is 6 in. thick, has a crown of 1 in. and is laid in 35-ft. sections, with ⅜-in. expansion joints. A 1:2:4 mixture with gravel as the coarse aggregate was used. The 6-ft. macadam shoulders, which are made of 2½ in. crushed limestone, taper to a thickness of 4 in., 4 ft. out, and from that line are feathered off to the edge of the side ditch.

Single-Track Roadway of Concrete Before Placing Macadam Shoulders

The 16 miles were divided into three separate jobs, but the lump sum bid was the lowest. Grading was begun in August, 1912. The roads at that time were ordinary country roads, with beds 10 to 20 ft. in width. The grading consisted very largely of widening the roadbeds to 20 ft. and reducing the grades. Little pay-haul was necessary. The excavation varied greatly, but the average cost was about 40 cents per cubic yard. Where there were sandfills they were covered with from 3 to 5 in. of clay. The limestone for the macadam was found in many places alongside the road, so that the transportation of the rock was only a matter of a short haul. Gasoline tractors with trains of 3-yd. wagons were used to haul the cement, sand and gravel. Two miles of concrete were laid with cement delivered 6 miles for slightly less than 20 cents a barrel. A statement was given out by the contractor to the manufacturer of the tractors that the sand and gravel were delivered for less than 6 cents per ton-mile. This figure was established on one section in which no difficulties were encountered. However, a transportation cost of 20 cents per yard-mile was easily maintained.

No sand cushions were used for the concrete pavement except on extremely wet portions of the road. The concrete was laid directly on the clay subgrade as prepared by the graders. Grooving of the concrete on the grades was tried, but it was not very successful and was abandoned. Considerable attention was paid to get the joints between the crushed-rock shoulders and the concrete flush. If the macadam is too high, water will back up and run down to subgrade through the joints; if it is too low, the concrete will project and break off under the action of the wagon wheels. Local engineers are planning experiments using road asphalt and cement grout along the edge of the macadam to determine whether an application of these materials will remedy this point of weakness in the design.

Recapitulation of the pavement costs showed that 18-in. concrete culverts cost about $50 each, placed in the road. The cost increased to $65 for 24-in. culverts, $85 for 30-in. culverts and $135 for 36-in. culverts. Including the cost of culverts, the grading expense averaged $1500 per mile. The macadam shoulders were an additional expense of about 25 cents per lineal foot of road. The 8-ft. concrete roadway cost 90 cents per lineal foot. The entire cost of 16 miles was, in round figures, $120,000, and is covered by the county's 3 per cent of the state quarter-mill tax plus the county appropriation and local contributions.

Future Plans

During the construction of the road last fall an amendment to the state constitution was adopted providing a one-mill tax for state-road purposes. This will give the state $1,500,000 a year to begin with, which will gradually increase as the assessed valuation increases. The law provides that no county shall get more than 3 per cent nor less than 1 per cent of the total fund. As there are 86 counties in the state, this leaves 14 per cent to be distributed at the discretion of the highway commission. Largely through the efforts of Dr. D. B. Pritchard, chairman of the Good-Roads Committee of the Winona Association of Commerce, a plan has originated for building an additional 80 miles of concrete roads covering most of the main-traveled highways in the county, connecting up every market town and providing an interurban and intercounty road system. As Winona has one of the four bridges across the Mississippi River between St. Paul and the Iowa line, this will also provide an interstate-road system.

The county board is now unanimously in favor of the plan. A committee has approached the highway commission and asked them to grant Winona County the full 3 per cent of the funds at the disposal of the state for the next ten years under the Elwell law. The plan apparently appealed strongly to the commission, which is impressed with its value as an object lesson to the rest of the state, and it is felt that the formal application will eventually be approved. Steps have been taken to have plans and specifications prepared for the additional 80 miles of roadway and the matter is then to be again brought before the highway commission. The Supreme Court has upheld the assessment feature and all other provisions of the Elwell law.

The Federal government has let a contract for a quarter of a mile of concrete road running through the U. S. Fish Hatchery, 6 miles below Winona, connecting at both ends with one of the county concrete roads.

The construction was originally carried on under the direction of O. B. Leland, late district engineer of the State Highway Commission, who was the untimely victim of an automobile accident last April.

Proposed Limitation of Height of Buildings on Fifth Avenue, New York

The Fifth Avenue Association, of New York, has presented to the New York City Building Heights Commission a statement regarding the height, size and arrangement of buildings which deals at length with the objections, dangers and disadvantages of very tall buildings in business streets. It concludes with the recommendation for legislation which would restrict the height of buildings on Fifth Avenue and within an area of 300 ft. east and west of the avenue to 125 ft. to the cornice line, with not more than two mansard roof stories additional, the restriction not to apply to steeples, domes, towers or cupolas of fireproof material erected for ornamental purposes. It is pointed out that the present tendency to the erection of high buildings, if encouraged, will finally ruin the beauty and charm of Fifth Avenue and make it dark and gloomy, and that continued congestion and the increase of cheap loft buildings will seriously damage it.

Appended to the statement is an unverified list of twenty-five cities in the United States and thirteen cities in foreign countries which restrict the height of buildings as follows:

1. To a flat height varying from 80 to 200 ft. or up to 12 stories.
2. Proportional to the width of the street and varying from one to two and one-half times its width.
3. Regulated by drawing a line upward at an angle from the edge of the premises of the opposite side of the street.
4. Based on the character of construction, or according to the zone system.
5. Proportional to the average of horizontal dimensions.

In foreign cities the flat-height limit varies from 43 ft. in Zurich to 150 ft. in Sydney.

Method for Designing Concrete Draft Tubes

Draft-tube design requires careful study, for on it depends to a large degree the efficiency of hydroelectric installations. Care must be taken that no abrupt changes in the velocity of the water take place. A flaring tube of circular cross-section would be ideal, but generally involves deep excavations, and in many cases the shape must, therefore, be flattened. The design of a draft tube of the latter shape presents a number of difficulties and a method derived by Herr R. Dubs, civil engineer, of Zurich, Switzerland, and published in "La Houille Blanche," by which these difficulties are overcome, is of value to engineers.

It is easy to determine the area at the top and at the end of a draft tube because the quantity and the velocity of the flow are known. The top area directly under the turbine can have but one shape—the circular; but the shape of the end area can be varied within a wide range, whose limits in a vertical direction are the depth of excavation and the water seal required and in a horizontal direction the thickness of masonry necessary to carry safely the weight of the power house.

The least amount of excavation is, of course, needed for a draft tube with a flattened rectangular end area. If in Fig. 1 the depth of the area is D, its width B and the radius of its fillets r, the area is

$$A = BD - 4\varepsilon$$

where ε is the difference between the square and the sector; but

$$\varepsilon = r^2(1 - \pi/4) = 0.8584\, r^2/4$$

which gives

$$A = BD - 0.8584\, r^2 \qquad (1)$$

Since for any area of the draft tube $A = Q/v$ where Q = quantity of flow in second-feet and v = velocity of flow in feet per second the end area can be determined.

The flow in the tube must decrease in velocity, and this is supposed to take place in such a manner that

$$sv^2 = k = \text{constant} \qquad (2)$$

where s is a distance as shown in Fig. 2. If, now, v_1 and v_2 are the velocities at the entrance and the exit respectively, s_1 and s_2 the respective distances from a certain axis and L the length of the draft tube measured on its center line, the following equations can be derived:

$$s_1 v_1^2 = k, \text{ thus } s_1 = k/v_1^2 \qquad (3)$$

and

$$s_2 v_2^2 = k, \text{ thus } s_2 = k/v_2^2$$

which give

$$s_2 - s_1 = L = \left(\frac{1}{v_2^2} - \frac{1}{v_1^2}\right)k$$

and

$$k = \frac{L}{\left(\frac{1}{v_2^2} - \frac{1}{v_1^2}\right)} \qquad (4)$$

k being known, s_1, s_2 and L are easily calculated.

When a particle of water is passing through the tube its velocity is expressed by $v = ds/dt$. This gives $v^2 = (ds/dt)^2$; $s\,(ds/dt)^2 = k$; $ds/dt = \sqrt{k}/\sqrt{s}$; $\sqrt{s}\,ds = \sqrt{k}\,dt$; and finally $dt = s^{\frac{1}{2}}\,ds/\sqrt{k}$.

Integrating this equation between the limits s_1 and s_2 gives

$$t = \frac{1}{\sqrt{k}}\int_{s_1}^{s_2} s^{\frac{1}{2}}\,ds = \frac{1}{\sqrt{k}}\,\frac{2}{3}\,(s_2^{3/2} - s_1^{3/2})$$

or

$$t = \frac{2}{3\sqrt{k}}[s_2\sqrt{s_2} - s_1\sqrt{s_1}] \qquad (5)$$

which expression indicates the time required for the water to pass through the tube.

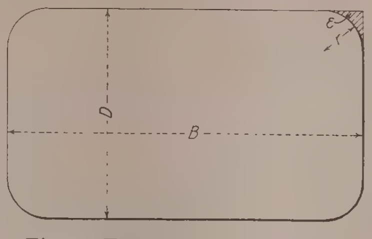


Fig. 1—End Area of Draft Tube

To be able to design the draft tube the following dimensions must be known: H, T, D_1, L and D. All of these are definitely known except H and T, which will have to be assumed. Two cases present themselves—in which T is equal to or larger than H.

Case 1:

$$T > H$$

$$n = \pi R/2 \qquad (6)$$

and

$$T + R = R\pi$$

which gives

$$T = (\pi - 1)R \qquad (7)$$

Therefore

$$\left.\begin{aligned} R &= T/(\pi - 1) = T/2.1416 \\ \text{and}\quad n &= \pi T/(2 \times 2.1416) = T/1.3634 \end{aligned}\right\} \qquad (8)$$

Case 2:

$$T = H$$

In this case the formulæ simply become

$$\left.\begin{aligned} R &= H/2.1416 \\ \text{and}\quad n &= H/1.3634 \end{aligned}\right\} \qquad (8_a)$$

The principal elements of the large curve being known, it is now necessary to calculate the dimensions governing the design of the smaller curve. From Fig. 3 it is apparent that

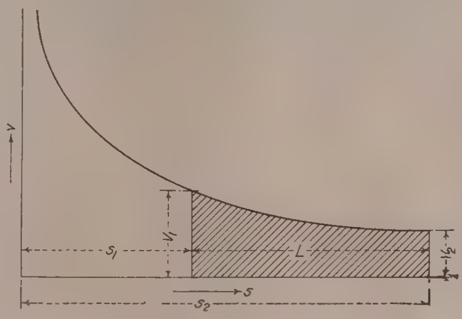


Fig. 2—Velocity Curve for Draft Tube

$$y = R\tan(\alpha/2);\; x = r\tan(\alpha/2);\; n = \pi R/2$$

$$z = n + \left(\frac{2\pi R}{360}\right)\alpha = \pi R/2 + \left(\frac{2\pi R}{360}\right)\alpha$$

$$n + y - D_1 - x = m$$

and

$$m + (2\pi r/360)\,\alpha = z - y - D - x$$

Eliminate m from the two last equations and the result is

$$n + y - D_1 + (2\pi r/360)\,\alpha = z - y - D + 2x$$

or

$$n + 2y - 2x + (2\pi r/360)\alpha = z - D + D_1$$

Putting into this equation the values of x and y, the expression becomes

$$n + 2R\tan(\alpha/2) - 2r\tan(\alpha/2) + (2\pi r/360)\,\alpha = z - D + D_1$$

Inserting now the values of z and n the equation finally reads

$$\pi R/2 + 2R\tan(\alpha/2) - 2r\tan(\alpha/2) + (2\pi r/360)\,\alpha = \pi R/2 + (2\pi R/360)\,\alpha - D + D_1$$

or

$$2r[\tan(\alpha/2) - \pi\alpha/360] = 2R[\tan(\alpha/2) - \pi\alpha/360] - D_1 + D$$

thus giving

$$\left.\begin{aligned} r &= R - \frac{D_1 - D}{2[\tan(\alpha/2) - \pi\alpha/360]} \\ m &= \pi R/2 + (R - r)\tan(\alpha/2) - D_1 \\ \text{and}\quad x &= r\tan(\alpha/2) \end{aligned}\right\} \qquad (9)$$

Should it happen that α = 90 deg., these formulæ become

$$\left.\begin{aligned} r &= R - \frac{D_1 - D}{2 - \pi/2} \\ \text{or}\quad r &= R - (D_1 - D)/0.4292 \\ m &= \pi R/2 + R - r - D \\ \text{and}\quad x &= r \end{aligned}\right\} \qquad (9_a)$$

When r, x and n are known the curve can be constructed without difficulty.

An Automobile Using Coal Oil was recently operated in Columbus, Ohio, with successful results, according to the "Industrial World." The device by which the cheaper oil is made to serve an ordinary gasoline automobile engine consists of a heavy copper cylinder or tank of about 20-in. diameter and 30-in. depth, divided by a head about one-third down from the top. Piercing the cylinder is a central shaft and several supplementary pipes and shafts. Air is drawn into the pipes by suction and forced down into the kerosene, to which has been added 10 per cent of gasoline. The air, impregnated with vapors of various hydrocarbon gases, is fed directly to the cylinders of the engine. No chemical examination of the gas generated has been made, nor does its chemical composition seem to be known, but it is reported as maintaining complete combustion and burning with a blue flame of extraordinary heating power.

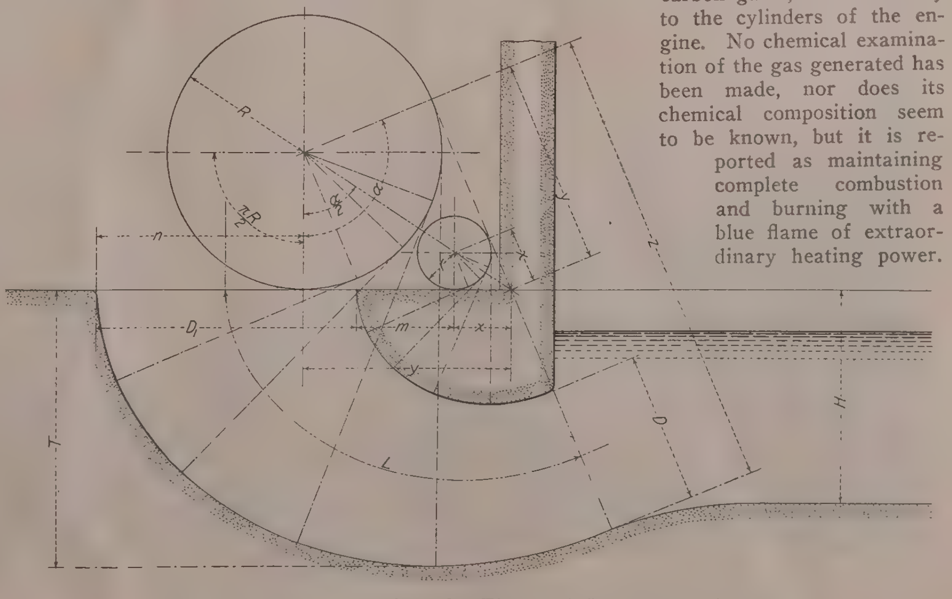


Fig. 3—Elements of Design of a Flattened Concrete Draft Tube

Company-Force Work on the Louisville & Nashville Railroad

Special Equipment and Results on Grade Revision and Double-Tracking

Mention was made in the first article on the Louisville & Nashville improvements, on page 97 of the issue of July 26, 1913, that the section from Athens, Ala., to the junction with the Southern Railway near Decatur is being done by company forces. This is a new departure for the Louisville & Nashville, and is being used somewhat as a test, careful cost records being kept. The section selected is about 12 miles long. The country is rolling, and the material is mostly earth, comparatively easy to handle. The cuts and fills are long, however, and the quantities are large, affording ample opportunity for saving by efficient methods and attention to details. A large part of the work is along the old right-of-way, where interference with regular traffic would be serious. The railroad company's general shops at Decatur are conveniently near for repairs and alterations to the equipment.

EQUIPMENT

The equipment in use at present consists of a steam shovel, a steam pile driver, a spreader, three work trains of ten side-dump cars each, and a concreting outfit. This equipment was bought especially for this work, and considerable study was made before it was ordered. The different machines have, furthermore, been modified and added to in a number of ways since they were put into service.

The steam shovel is a 110-ton Atlantic, built at the Paterson works of the American Locomotive Company. It originally had a 3½-yd. dipper, but after this had been tried out it was replaced by one with a capacity of 4½ yd. A small locomotive tender was added to supply water and coal, but for this has been substituted one of the largest type in use on freight locomotives of this company, with a tank capacity of 8000 gal. The latest addition is the small caboose shown in one of the illustrations. This contains a complete workshop, and all ordinary repairs to the shovel are handled without taking it away from the work. A change in gears has also been made whereby the propelling power of the shovel has been doubled.

Thirty steel 16-yd. air-dump cars, made by the Kilbourne & Jacobs Manufacturing Company, are used. As previously stated, they are handled in ten-car trains. Ordinary locomotives of a small size haul them, but in order to obviate frequent runs for water without carrying the load all on one large tender, and thus necessitating expensive track for work-train service, each locomotive carries two small tenders with a combined water capacity of about 8500 gal.

A Jordan spreader and a Bucyrus pile driver complete the grading equipment. The spreader is shown in action in one of the illustrations. Its collapsible arms are operated by compressed air from the work-train locomotive, and the outfit will spread the material to the width of a track on either side of the operating track. The steam pile driver, the tower of which is hinged to allow it to be folded back to meet clearance requirements when in transit, is also at the present time equipped with one of the large tenders and a workshop caboose.

Self-Propelling Steam Shovel of 110 Tons Capacity with 4½-Yard Dipper

Note large tender and caboose fitted up as workshop

TANNER CUT

Beginning at Athens, the old grade is being revised and a second track added for a distance of about 8½ miles. A new double-track line then turns off to the westward, forming a new junction with the Southern Railway, over which the Louisville & Nashville Railroad operates into Decatur, about 1 mile south of the old one. About 1¼ miles south of Athens a cut containing 75,000 cu. yd. will be made with the new grade 8 ft. below the existing track at the summit, and 3½ miles south of Athens the Tanner cut begins. For a distance of nearly 2 miles the old grade is being cut down, the maximum amount being 24 ft.; and as the old track was in about a 6-ft. cut some of the way, the new double-track cut is 30 ft. deep at the highest point, and about 300,000 cu. yd. are involved.

The material is being distributed both ways to take up sags. About 2½ miles south of the cut Swan Creek is crossed by a trestle 350 ft. long. The new track is being carried across by a trestle of the same length, with creosoted piles and a ballasted deck. The new line is largely embankment across a swamp. Only a comparatively small amount of concrete is required on the entire 12-mile section.

The shovel began work at the Tanner cut Jan. 4, 1913. Traffic on the original main track was not molested until the first cutting by the shovel had been taken out to elevation about halfway between the original and revised grades. A track for main-line traffic was then laid on the extreme west side of this cutting, and it will be left there until the cut is completed, the first cutting having been taken out of sufficient width to permit the double-track cut to the final grade being constructed without again interfering with the main-line traffic. The slope on the side next to the temporary main line will, however, be left somewhat steeper than the permanent slope, but after traffic has been diverted to a track on the permanent grade the temporary track will be removed and the material supporting it taken out, the cut being finished somewhat wider than the standard section on that side to permit the construction of a wide ditch to handle the drainage through the cut, which, as stated above, is practically 2 miles long with the grade descending throughout its length. The material encountered in this cut is mostly earth, but there is hard material enough to warrant the belief that with a less powerful shovel much of it would have to be loosened by blasting. As it is, no blasting has been required; but it might be mentioned that it is necessary to replace the manganese-steel dipper teeth about every six weeks.

Early in May the shovel finished the first lift and regular traffic was shifted to the temporary track in the cut. At that time two of the work trains were carrying material north

Work Train of 16-Yard Steel Air-Dump Cars Being Emptied Simultaneously

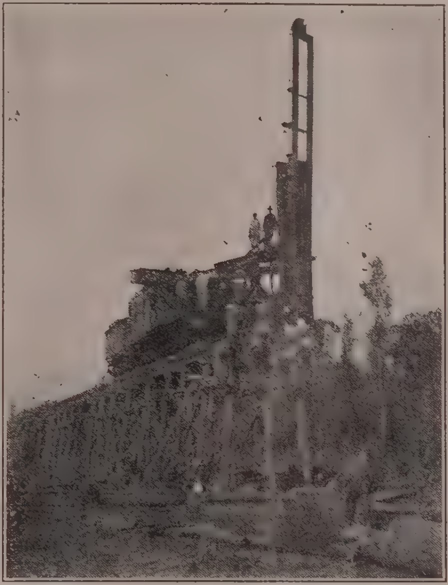
Locomotive carries two tenders for water and supplies and dumping from the main track and the other was carrying it south and dumping from a siding, temporary passing sidings being so arranged that the shovel would be idle waiting for cars a minimum of time.

Work of Shovel and Work Trains

The shovel, with the 3½-yd. dipper, was put into service June 10, 1912. At Christmas time the men were given a vacation and the shovel was in the Decatur shops for repairs from Dec. 22 to Jan. 4. Aside from this there had been, up to May 1, only two days, exclusive of Sundays, in which it had not been in service. The records give the cause and duration of each delay of from 0.16 of a day upward, and show that the accumulated monthly losses in time for repairs were from 6½ hours to 3½ days. When the shovel was first put into service there were only ten dump cars available, and time lost by the shovel in waiting for cars was necessarily large. Since the third work train has been in use the records show that the loss of time waiting for cars is ordinarily from 15 to 20 per cent of the total day.

It is found that the average load per 16-yd. car is about 12.6 to 12.8 cu. yd., pit measure. Up to April 1 the largest yardage handled in any month was that for October, 1912. During that month of twenty-six working days 4507 carloads, or 55,616 cu. yd., of material, pit measure, were taken out. This was all classified as earth except 3781 cu. yd. of loose rock. The average daily yardage was 2139 and the maximum 4319. During this month the average working day was 10 hours 23 minutes, although in other months this was somewhat lower. The cost per cubic yard of loading was $0.0365. The unit cost ran lower two other months, and also ran as high as $0.0624; the average seems to be about $0.045.

The work-train haul in miles averaged by months varies from less than 1 mile to more than 4 miles. The time consumed in percentage of the total day has been itemized as follows: Loading, unloading, spreading, hauling, delay by trains and delays at shovel. The loading consumes from 20 to 30 per cent of the time and the hauling from 15 to 40 per cent, depending largely on the length of haul. Unloading takes, on the average, only about 4 or 5 per cent of the day, and spreading about twice that amount. Most of the lost time can be attributed to delay by trains—running from 15 to 20 per cent. The delays at the shovel, as a rule, use less than 10 per cent of the total day.



Pile Driver in Action

In the month of October, when the maximum yardage was handled, the average haul was only 1.43 miles, and more time was spent in loading than in hauling, the two amounting to 61.54 per cent of the total time. Unloading and spreading together took but 9.12 per cent, but delays at the shovel consumed about 13.64 per cent and delay by trains, of which there were 401 revenue trains during working hours, 15.70 per cent. It should be stated, however, that during this month only a small portion of the hauling was done over the main line, as all loading was done while trains were standing on temporary tracks and all material unloaded while trains were clear of the main line. The percentage given for delays by trains includes interference of one train with the other in making its move from the shovel to the dump.

Unit Costs

The cost of work-train service for October for loading, unloading and spreading was $0.0203 per cubic yard. This gives a figure of $0.0137 per cubic yard per mile haul. In other months when the average haul was longer the cost per cubic yard ran as high as $0.045, but the cost per cubic yard per mile figured lower, averaging about $0.01. Of the $0.0203, $0.0076 was chargeable to delays by trains.

Spreader Distributing Material for Low Earth Embankment

Progress of Work

Most of the filling is being done from temporary trestles of a type of construction capable of supporting the heavy track pile driver shown in the illustration. During September, 1912, 16,000 lin. ft. of piles were driven and 4150 lin. ft. of trestles were built. Up to April 1, 1913, about 18,000 lin. ft. of temporary trestle had been built, at a total cost of about $40,000, or $2.50 per linear foot. In addition 480 lin. ft. of permanent trestle were built at a cost of about $4,200, or about $8.80 per linear foot.

Figures have also been deduced to give the monthly average cost per cubic yard for temporary trestles and for track work. These figures necessarily vary within rather wide limits, as the trestles built and the track work done in any month may not in any way represent the amount required in a month, and the cost can be distributed to the proper yardage only approximately. The cost per cubic yard of trestle work ranges from $0.055 to $0.128, and that of track work ranges from $0.026 to $0.061. Summing up the four main items for the grading—shovel expense, work-train service, track work, and temporary trestles—the total cost per cubic yard is found to average about $0.25, including all haul on the material, although the average for the month in which the most grading was done—October, 1912—was only $0.20. It might also be mentioned that in the month of December, 1912, when practically a third of the material handled was loose rock, the average cost per yard was less than $0.23.

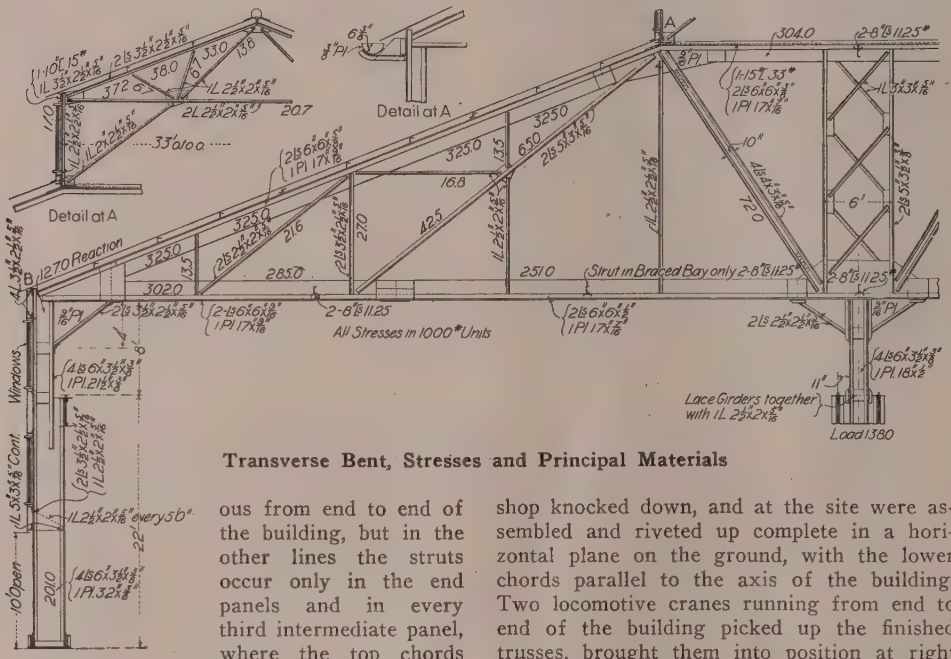
In obtaining the various costs given above care was taken to include every item that should enter into them. The steam-shovel figures include interest, depreciation and repairs on the shovel; interest and depreciation on the water-service equipment and an item for the water itself; coal, oil and freight; wages of engineer, craneman, fireman, pit hands and night watchman, and labor in coaling; interest, depreciation and repairs on the camp cars for the shovel crew, and superintendence. The work-train expenses were obtained in a similar manner, the item for wages including those of the engine crews, train crews, block operators, night watchmen, dump men and spreader men. In the same way the trestle and the concrete costs were analyzed, interest and depreciation being figured on the small tools as well as on the main equipment.

The work is being done under the general direction of Mr. John Howe Peyton, chief engineer of construction. Mr. C. H. Crawford, resident engineer, is in immediate charge, reporting to Mr. Theodore Speiden, Jr., roadmaster, who in turn reports to the chief engineer of construction.

The Indo-Ceylon Railway is to be opened with much ceremony by the Viceroy of India and Governor of Ceylon in January, 1914. Rails have been laid, according to dispatches to the "Consular and Trade Reports," as far as Talaimannar, the terminus on the Ceylon side. At this point two large jetties have been built, at the end of which warehouses and customs houses are to be erected. A three-track line connects them with the shore. The concrete causeway and roller bridge across the Qaumben Channel have been completed.

Long-Span Rolling-Mill Roof Truss

The 780 x 140-ft. steel building for the 8, 12, 16 and 20-in. merchant mills of the Republic Iron & Steel Company, of Youngstown, Ohio, has unusually long roof trusses spanning the full width of the building without intermediate supports, and carrying at the center two lines of runway girders for the 7½-ton electric traveling cranes, thus leaving the entire floor area unobstructed by interior columns. The building has thirty-one panels about 25 ft. long, and each of the intermediate bents consists of two columns and a roof truss, as shown in the accompanying sectional elevation. The bents are braced longitudinally by the plate-girder crane runway, 30 in. deep, by the purlins, by lines of lattice girder struts connecting the columns at the eaves and 10 ft. clear of the ground, by two lines of intermediate lattice-girder struts and by five lines of T-shaped longitudinal struts made of two 8-in. channels connecting the bottom chords and one at the center points of the top chords. The two center lines of these struts are continuous from end to end of the building, but in the other lines the struts occur only in the end panels and in every third intermediate panel, where the top chords and the upper portions of the columns are X-braced. In the unbraced panels the lines of struts are continued by ¾-in. tension rods. The bottom chords are braced by diagonals crossing six roof trusses.



Transverse Bent, Stresses and Principal Materials

A hipped monitor 33 ft. wide, with sliding windows 5 ft. high in the clerestories, surmounts the roof, and there are two similar continuous lines of windows in each long side of the building. The walls are framed with single 5 x 3 x 5/16-in. door-post angles and 3½ x 2½ x 5/16-in. girt angles. The purlins are single 4 x 3 x 5/16-in. angles trussed with upset rods ⅞ in. in diameter. The roof is proportioned for a load of 40 lb. per square foot, plus dead load, plus crane loads, and both it and the walls, except at the window and door spaces, are sheathed with corrugated black N. B. iron of Nos. 20 and 22 gage respectively. On all sides of the building the spaces between the wall columns are open to a height of 10½ ft. above the floor level, and in one end there are two wide doorways 22 ft. high.

An unusual type of roof truss was adopted, with a very wide vertical post at the center. This is riveted to a special section of the web plate of the bottom chords, which projects beyond the bottoms of the flange angles, where it receives a short vertical post bearing the center lines of runway girders. The top and bottom chords have T-shape cross-sections, made with a pair of angles and a vertical web plate wide enough to receive the riveted connections of the web members of the truss without requiring gusset plates. The end sections of the top-chord and bottom-chord web plates are made integral with the upper parts of the column webs, thus forming a rigid connection equivalent to a heavy kneebrace between the column and roof truss. The columns are virtually plate girders 32 in. deep up to the top of the runway girders, which are field-riveted through their webs to the column flanges. At this elevation the column web is spliced to the narrower web of the upper section, the inside flanges of which are continued down below the runway girder to reinforce the splice, the outside flanges being continuous to the top of the column. Each column is anchored to its pier by four 1¾-in. bolts 8 ft. long. These gave it temporary stability after its erection before the roof trusses and braces were assembled to it.

The trusses were shipped from the bridge shop knocked down, and at the site were assembled and riveted up complete in a horizontal plane on the ground, with the lower chords parallel to the axis of the building. Two locomotive cranes running from end to end of the building picked up the finished trusses, brought them into position at right angles to the axis of the building and hoisted them with tackles attached near the hip joints, holding them until bolted to the columns and braced in position. The cranes had capacities of 22 and 30 tons respectively, and their 60-ft. booms were lengthened with temporary 10-ft. steel extensions. The cranes erected all of the steelwork except some light material hoisted with whip lines. A compressor car having a capacity of 250 cu. ft. of air per minute formed part of the erection equipment and furnished air for the pneumatic hammers, which drove about 27,000 field rivets. The steelwork was fabricated and erected by the McClintic-Marshall Construction Company, of Pittsburgh.

Framework of Building, with Derricks for Erection of Steelwork

Industrial Education in Berne, Switzerland

By F. P. McKibben, Professor of Civil Engineering, Lehigh University (on Leave of Absence Abroad)

Preparation for entering the industrial ranks in Berne must be considered seriously by the boy who undertakes it. It is no child's play. On leaving the primary schools, in which attendance is compulsory between the ages of six and fifteen, a boy wishing to learn a certain trade has two paths before him. He may learn it either by studying in an industrial school or by entering directly upon his work in a shop, where he is classed as an apprentice. In either case he is required by law to follow a system of instruction, as will be explained later.

Berne has five schools which offer special training for industrial pursuits—Gewerbeschüle, Lehrwerkstätten, commercial school for girls, commercial school for boys, and industrial school for girls. Only the first two of these will be considered here.

The Gewerbeschüle furnishes instruction to apprentices working in industrial establishments after they have finished the required primary-school work. It gives not only the instruction useful and directly applicable to their trades, but requires that they pursue general studies more advanced than those taught in the primary schools. The canton law requires attendance at this school seven hours a week for either three or four years, depending on the trade. Compulsory attendance at the Gewerbeschüle, where some phases of the industrial training can best be taught, is, therefore, an important provision of the apprentice system. That there is a demand for such training is shown by the attendance. In the summer course of 1911 there were 729 students enrolled in 49 classes; in the winter course following, 955 students in 58 classes.

Each apprentice is given three and a half hours' instruction a week in some branch of his industrial pursuit and three and a half hours in general studies, such as mechanics, algebra, trigonometry and modern languages. One example will illustrate. Boys working in machine shops receive three and a half hours of special training in mechanical drawing, and excellent drawings in great detail are made of pumps, electrical apparatus and other small machines; apprentices in electrical shops make very creditable detail drawings of a telephone receiver; apprentices from carpentry and furniture shops make drawings of objects in wood. And thus the work is adapted to the needs of the individual, which is admirable from a pedagogic standpoint.

CHARACTER OF INSTRUCTION

The work covers a wide range of industrial pursuits, but drawing and classwork in general subjects constitute the greater part. The following partial list of classes shows that an attempt is made to assist the apprentice in whatever trade he may engage—carpentry, masonry, decoration of buildings, architectural ironwork, bookbinding, plumbing, carriage building, leather working, printing, machinery manufacture, electrical equipment, cabinet-making, gardening, and even wigmaking. In the course in gardening instruction is given in laying out gardens, including a little surveying. Generally, the apprentice is not given instruction in the manual part of his work; for this training, I presume, the school system depends on the industrial plant where the apprentice is employed.

There is no charge for instruction, and nearly all materials are furnished gratis by the school. Neither is there any pecuniary remuneration for work done by the apprentices, for they are compelled by law to attend, and the products of their activity are such that no income can be derived from them. Most of the apprentices are from 16 to 20 years of age, but there are a few students, not apprentices, attending the school for special instruction who are much older.

Apprentices in machine shops are paid by their employers a wage varying from 5 to 8 francs ($1 to $1.60) per week during the first year to from 12 to 18 francs ($2.40 to $3.60) per week during the fourth year of their apprenticeship. Upon completing their apprenticeship and their three or four years' course of study in the Gewerbeschüle many of the men leave the shops where they have learned their trade and seek employment elsewhere. But they generally follow the kind of work for which they have prepared themselves. A few take the course in the Gewerbeschüle as a preparation for the higher technical schools. I am informed that the wage of machinists in the shops of Berne varies from $6 to $12 per week.

Five teachers devoting all of their time and about fifty others who give a little time for instruction either in the evening or day classes constitute the faculty. Of the latter group some are teachers in other schools of Berne and some are tradesmen who are engaged to give special instruction. The cost of operating the school is approximately 1,000,000 francs ($20,000) per year, borne collectively by the city of Berne, the Canton of Berne and the Swiss Confederation.

The Lehrwerkstätten is a manual-training school for boys who have finished their primary-school courses and desire to enter the trades by means of the school rather than by the apprentice system. In this school a boy spends three years studying and working in excellent shops if he desires to become a cabinetmaker, a plumber or a tinsmith, and four years if he wishes to master the machinist's trade. During these years he spends part of the time in general studies—mathematics, physics and modern languages—and the remainder in shopwork and drawing. Except for a two weeks' vacation in the summer and a week at Christmas, the work is continuous throughout the year, and, moreover, the day's work is from 7 to 12 in the morning and from 1:30 to 6 in the afternoon, or a total of fifty-two and a half hours per week. In the first three years the machinists devote an average of eleven hours per week in the classroom to general studies and drawings and an average of forty-one and a half hours per week to shopwork. In the fourth or last year they spend practically all of the fifty-two and a half hours per week in the shop. The three-year courses for architectural-iron workers, woodworkers, and tinsmiths are somewhat similar to the first three years of the machinists' course.

Certainly a boy who starts in on this schedule at the age of 15 years and sees it through to the end of three or four years must take his training seriously. It is a pretty rigorous course. In the architectural-iron workers' class there are 30 boys; woodworkers, 30; machinists, 68; plumbers and tinsmiths, 20—a total of 148 boys, mostly serving their apprenticeship in this school instead of in the industrial plants.

WORK DONE BY STUDENTS

Students in the woodworking department are making tables and bookcases which are well built and beautifully finished, and those in the metal-working department are making machines and parts of machines. It is interesting to learn that some manufacturing establishments send certain parts of machines to be finished at the school, and that in other cases the school supplies dealers with objects for sale. For example, one dealer buys each year from thirty to forty large oil cans, provided with hand pumps, which are made by the classes in sheet-metal work. That the work is satisfactorily done is shown by the fact that in some cases this relationship with commercial interests has existed for a number of years. The drawing courses are well adapted to the needs of the different classes; thus the sheet-metal workers receive instruction in projection drawing and its application to development of surfaces.

The Lehrwerkstätten course differs from that of the Gewerbeschüle in that for three or four years students devote practically all of their time to the school course, whereas in the Gewerbeschüle the apprentices in trades spend only seven hours per week in the school.

Students in the Lehrwerkstätten receive payment for their work during three years. The machinists' schedule will suffice for illustration. In the first year they receive no pay; in the second year a minimum of 25 centimes (5 cents) and a maximum of 35 centimes (7 cents) per day; third year, 50 to 65 centimes; fourth year, 75 to 90 centimes. The exact amount received between these minimum and maximum limits is determined by the student's scholastic standing. All materials are furnished by the school, which is maintained collectively by the city, the Canton of Berne, the Swiss Confederation and by sales of manufactured products finished by the students. The latter item amounts to about 90,000 francs ($18,000) per year, whereas the entire cost of operating the school is about 212,000 francs per year. There are twelve teachers and four heads of departments who devote all of their time to the school work, the former receiving 4200 francs per year each and the latter 6000 francs per year each. In addition there are three assistants and an administrative corps of four persons.

It seems to me that the Lehrwerkstätten should produce good men for the industrial pursuits. It is a busy place and there appears to be no time lost within its walls. The same is equally true of the Gewerbeschüle.

Experiments on Moving Loads on Highway Bridges

The results of several years' tests and experiments on highway bridges under moving loads made, recorded, analyzed and discussed by Prof. F. O. Dufour, assistant professor of structural engineering in the University of Illinois, are presented in a paper published in the June number of the "Journal" of the Western Society of Engineers. The investigation was made because it was believed that impact requirements of specifications, were not justifiable and that the limitations of fabrication and erection introduced conditions which modified considerably the theoretical stresses. Diagrams were made from the observations with a Frankel deflectometer and a Turneaure type extensometer, which multiplies the deformations about fifty times in a gage length of 48 in.

From four to six records were made for each member of each of seven bridges in Champaign and LaSalle Counties, Illinois. These included one 55-ft. reinforced-concrete-floor pony-truss span, one 70-ft. plank-floor through-truss span, one 78-ft. reinforced-concrete-floor through-truss span, one 105-ft. plank-floor through-truss span, one 110-ft. reinforced-concrete-floor through-truss span, one 144-ft. reinforced-concrete-floor through-truss span and one 160-ft. plank-floor through-truss span, the clear width of roadway varying from 14 to 16 ft.

CONCENTRATED LOAD

The concentrated load and corresponding average uniform load per lineal foot of wheelbase were as follows:

Concentrated load	Av. per foot in pounds
1700-lb. horse and buggy	113
4300-lb. automobile	430
8800-lb. team and load	490
12-ton farm engine with 13-ton stone wagon	1500
8-ton street roller	1700

The impact tests indicate that there is a critical speed; that the lighter the load the greater the percentage of impact; that for equal spans concrete-floor bridges have considerable less impact; that extensometers and deflectometers do not indicate the same maximum percentage of impact under the same loading and speed; and that there is not apparently any relation between impact percentage and span lengths. The impact increases as the loading decreases. Impact percentage is less in built-up members than in simple sections, and varies according to the location of the member.

The impact percentage for a horse and buggy is highest for a trot and is much reduced for a walk and a gallop. The impact percentage for plank floors increases considerably as the speed increases, but little or no impact is shown by the diagram for the reinforced-concrete-floor spans, the stresses in the truss members of which are very small, which is considered to be due probably to the fact that the floor distributes the load over the entire span. Curves plotted for the observed and computed impact stresses from an 8-ton roller on the 78-ft. reinforced-concrete-floor span indicate that the actual impact stresses are very small and the impact formula is practically valueless.

STRESS AND DISTRIBUTION OF FLOOR LOAD

Results indicate that the stress is unequally distributed over the section of compression members and that tension members composed of bars do not have the bars equally stressed.

Test diagrams of loads on concrete floors show that one joist carried 21 per cent of the total load, or 42 per cent of the wheel load. A single-plank-floor test showed that the joist carried 44 per cent of the total load, or 88 per cent of the wheel load.

In nearly all cases the computed stresses were less than the actual stresses and the difference was much greater in reinforced-concrete-floor spans than in plank-floor spans. In the former, few actual stresses exceeded 50 per cent of the computed stress.

CONCLUSIONS

Conclusions to be derived from the results of the experiments are:

1. That built-up members are superior to single shapes.

2. That light loads produce the greatest percentage of impact, but that the light-load static stress, plus the light-load impact stress, does not exceed the static stress for which bridges are usually designed.

3. That reinforced-concrete-floor structures are much superior to the plank-floor structures.

4. That on account of the fact that the stresses produced by heavy loads in reinforced-concrete-floor structures are very small, the impact percentages were difficult to determine and that the values given are probably far in excess of the actual, which appeared in the majority of cases to be zero.

5. That actual stresses in reinforced-concrete-floor bridges may be assumed as one-half the computed static loads.

6. That considering conclusions 1 to 5, we may safely neglect impact in reinforced-concrete-floor bridges, and that we may assume each joist to take 20 per cent of the wheel load or 10 per cent of the axle load.

The Illinois State Highway Commission has decided that impact may be neglected in proportioning reinforced-concrete-floor bridges with an earth cushion and proportioned for a live load of 80 lb. per square foot of floor surface. This is believed to provide sufficiently for various secondary stresses.

THE PETER THE GREAT BRIDGE across the Neva at St. Petersburg has two arch truss spans of about 435 ft. separated by a double-leaf bascule span. The main spans have suspended floors and are supported on very heavy masonry piers with foundations on 40 x 145-ft. steel pneumatic caissons. The bridge is a monumental structure, with ornamental details and towers for the bascule span. The bottom chords of the arch spans are the principal truss members and are of unusual construction, having a deep H-shape cross-section consisting of two built channels connected on the center line by a continuous horizontal diaphragm. It is built up entirely of plates and angles, the diaphragm having three thicknesses and the channel webs four thicknesses of plates. The latter are reinforced on the outer faces with five pairs of longitudinal angles riveted together back to back with the vertical flanges, together covering the entire face of the web between the flange angles. The top and bottom flanges are made of single outside angles, with their horizontal flanges reinforced by three cover plates.

Letters to the Editor

Code of Ethics of the Pacific Northwest Society of Engineers

Dear Sir: In your issue of July 12, 1913, page 50, reference is made to the Pacific Northwest Society of Engineers adopting a code of ethics for engineers.

Early this year a committee was appointed to draw up and present a code to the society at its annual convention, which was held on June 20 and 21. The code presented by the committee, being the one printed in your paper, was adopted, with the provision that before final adoption, or rejection, it be submitted by letter ballot to each member of the society. The ballot will be taken some time this year.

Evidently the entire proceeding was not made clear to your representative.

JESSE A. JACKSON,
Secretary, Pacific Northwest
Society of Engineers.

Seattle.

Unfairness to the Contractor

Dear Sir: The writer, being an engineer, has had occasion to draft specifications for work of varying degrees of magnitude and importance, and has, as yet, made no radical departure from the stereotyped forms which tradition has imposed upon engineers, due partly to a natural hesitation to depart from those standards which have so long seemed satisfactory to engineers of high standing and partly to the feeling that he could devise no better form to fully protect his clients.

Of late, having been obliged by circumstances to look at matters from the standpoint of the contractor, some features have seemed a rank injustice to men of that calling. It has seemed that the engineer is so zealous to protect his client that he entirely loses sight of what is due the other party to a contract. I think this is not wholly due to a distrust of contractors but to a fear that he may not properly safeguard the interests of his clients. It has been said that an engineer can break a contractor; to do this it is not necessary to be dishonorable, it being sometimes sufficient merely to enforce the letter of the contract rather than the spirit of it. While the engineer is paid by—we will say for a convenient term—the owner, he is really a servant of both parties to the contract, whose function is to enforce safe methods of work and to the best of his ability carry out that which was intended in the contract. In many classes of work it is impossible at the time of writing the specifications to foresee what may occur, and in that case the tendency is to place all the risk on the contractor, and in cases of doubt in interpretation of the contract "the decision of the engineer shall be final and binding." Rather than impute any dishonest motive to the engineer, I am inclined to think that he is too fearful of being considered dishonest or unprofessional, and he thus loses all sense of perspective, and his vision is so narrowed that he cannot fairly decide a question in which his employer is involved. The contractor should have the right of appeal from the decision of a man in the pay of the other party to the contract.

Again, "Any alteration in the lines or grades may be made after the work has commenced, if in the opinion of the engineer it becomes necessary." Some similar phraseology to the above is inserted in nearly every contract, and it fully protects the owner. But is it fair to the contractor, who "has viewed the premises and carefully studied the plans and specifications"? The writer thinks not, and has an idea that both parties to a contract should be protected, and in case of disagreement reference should be made to an arbitration board to decide the merits of the question. I shall hesitate hereafter to draw up a contract which binds absolutely the contractor and is very elastic as applied to the owner.

But there is another injustice to the contractor, though of minor importance. Many jobs are advertised for which there is but one copy of the plans, which the contractors are supposed to look at and absorb, and then, after going over the ground with some person utterly incompetent to answer relevant questions, repair to the hotel and figure the bid without plans to work with. Surely, on a job of any consequence the engineer should see to it that every contractor is provided with a full set of plans, giving all the information available. I have put in bids on six jobs in the last three months, running between the figures mentioned above, and in every case the contractors were deprived of the personal use of plans, and there were some special and extraordinary features in each case. To submit an intelligent bid, one should have plans, profiles, results of borings or test pits, if any, right on the little washstand of the country hotel, where we have all sat even into the small hours of the morning compiling bids.

GUY H. CHASE,
Of Parker, Bateman & Chase,
Civil Engineers.

Fitchburg, Mass.

Engineering Standards and Architectural Requirements

Dear Sir: In your editorial on the above subject discussing the co-ordination of architects and engineers, published Aug. 2, there are several points which may be enlarged upon.

In connection with industrial plants it might be well to state that the larger and more complicated ones are now being designed almost entirely by engineers, as the engineering work in connection with the same is much larger than the architectural work, and that on this class of work where architects are employed they are under the engineer. This seems to me is as it should be, as the particular profession whose work preponderates should have charge of the same.

Regarding the compensation for engineers, I have thought for a long while that the contract for professional services between the owner and the architect should specify a definite amount, perhaps a lump sum, but preferably a percentage, to provide for the engineer's services, and that this part should be used for no other purposes. In other words, all inducement for the architect to save money on the engineers' services should be removed. The best architects are not now trying to save money on engineers' services; but I think you will appreciate that the temptation is present for some firms to do so.

You also mention alternative schemes for performing the engineers' work—first, in large offices the architects can employ specialists of their own; second, in smaller offices outside consulting engineers are employed. I think that the first scheme, even in the largest offices, has decided disadvantages for the following reasons:

(1) Very seldom or ever does the architect

secure as good a man as he could secure if he employed a consulting engineer. The best grade of men will not work permanently at a salary in an architect's office.

(2) Where the architect employs an engineer at a salary that engineer has experience only in the particular work which passes through that one office, while a consulting engineer that does the work for numerous architects derives experience in different methods of doing work, and also in a much greater variety of structures.

In addition to the above, I think the architect has a considerably better basis for determining who is a good man from among the consulting engineers than from the men who would ordinarily accept a position on a salary, as in general the consulting engineer has a reputation which is a good index of his merit, while the man working at a salary ordinarily has not secured any particular reputation from which an architect can judge of his merits. To illustrate this I will state that a prominent architect in New York hired an engineer from one of the structural-steel concerns. After trying him for about six months he discharged him because he had failed to do his work in a satisfactory manner. The engineer himself was a good man, but he found conditions so different in an architect's office from what he had known before that he was utterly unable to adjust himself to his new position.

The point in your editorial concerning the use of extra steel to accommodate particular architectural treatment is excellent. To be a successful engineer one must always keep the viewpoint of the architect in mind. It is the duty of the architect as much as it is of the engineer to produce the desired result at the minimum of expense. The engineer, however, too often forgets that the decorative features of a building form an essential factor contributing to its success, and these should not be overlooked any more than the strictly utilitarian features. In my experience I have never known a case where a careful study of a problem with an architect did not result in securing what they desired and also securing first-class and relatively economical construction.

H. G. Balcom,
Of Balcom & Darrow, Engineers.
New York City.

Getting Along without an Engineer

Dear Sir: I am sending you herewith a shining example of home-made plans and specifications. It is an extract from a published advertisement for bids on three "cement bridges" for a Western board of county commissioners. The description not only is an advertisement, but constitutes, as well, the plans and specifications." "No engineer was employed," the writer was informed, "because of the expense."

Will some of your readers kindly exercise their wits and guess as to what kind of a cement bridge the commissioners had in mind, and also hazard a guess as to the cost?

The "plans and specifications" follow:

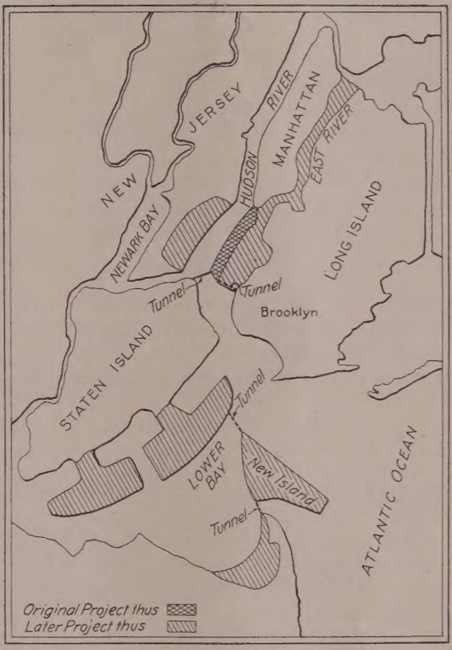
"One cement bridge 60 x 20 x 8 ft., at a point about 40 rods south of the railroad crossing at the west end of the Town of A——, on State Highway No. 19—that is to say, the wings shall be 60 ft. in length from bank to bank, the tops of such wings to be 2 ft. higher than top of the arch; average openings to be 8 ft. high, the arch to be 20 ft. wide, the base to be 12 in. at the bottom and 6 in. at the top of said wings; the arch to be 12 in. thick throughout, except the arch is to be set on a solid foundation 2½ ft. thick, and with a floor to such arch 5 in. thick, and spillways on each side of the arch of same thickness and 24 in. wide on each side of arch; the contractor to set the railings complete. County will supply all railings, cement and reinforcement, at the Town of A——."

Colorado Springs, Colo.

Contractor.

A Really Greater New York

Dear Sir: The dock problem in New York has been becoming more and more serious every year, and many requests for dock privileges have been and are daily refused for want of space. All the proposed remedies were of such doubtful or trivial value that I was constrained, some two years ago, to publish the first part of my proposed solution, which was, in short, as follows:



Proposed River and Harbor Changes

1. To build sea walls from the Battery, half a mile apart and about 4 miles long, thus forming a new neck of land of 2¼ sq. miles area, leaving a channel of about 1¼ miles between the new Battery and Staten Island.

2. To tunnel to Staten Island and South Brooklyn.

This plan would at once add 8 miles of new docks. The docks could be 1000 ft. or more in length, and equipped with all modern machinery.

Staten Island is now assessed at $50,000,000, and as soon as this scheme is carried out the value will be increased to at least $500,000,000, and the value of the 2¼ sq. miles of new land and docks would easily be $1,000,000,000.

But I realize that this plan, vast as it is, would also be inadequate—in fact, Commissioner Tomkins said that the minute the new land and docks were made available they would be so enormously valuable that they would immediately be snapped up, and the congestion would be just as bad as ever.

I then published the second part of the scheme, which I had been holding back—the second part being the formation of a new and larger bay between Staten Island and Sandy Hook and at the same time reclaiming some 40 sq. miles of valuable waterfront land. To my surprise, the greater plan did not frighten people as much as the first one—in fact, three quarters of the opposition aroused by the first disappeared on the publication of the second plan.

Since then the plan has been thoroughly discussed by many captains of industry, and the suggestion has been made by some very eminent men that this plan would be much improved by the addition of a plan—first proposed some sixty years ago—of reclaiming the East River for city purposes and constructing a new East River between Flushing and Jamaica Bays.

This, of course, adds enormously to the value of my plan, but could not be considered by itself—for to fill up the East River would make the harbor congestion far worse than it is now—unless Staten Island and Sandy Hook are connected up as shown in my plan, which provides for an unlimited extension of docks.

Needless to say, the carrying out of this plan—which is only a question of time—would create the greatest boom that ever occurred in this country.

For years to come it would involve the expenditure of not less than $30,000,000 a year for labor, and the same amount for material; cause every transportation line to be kept busy bringing material from every part of the country, and would keep every industry in the city busy housing, feeding and clothing, etc., the laborers.

This would permit of a park boulevard from Sandy Hook to Long Island Sound and a wonderful system of rapid transit lines. It would solve the Brooklyn Bridge and tunnel problems.

It would also permit of the construction of a great trunk sewer from Yonkers and New Jersey, down through Staten Island, past Sandy Hook, far out into the ocean. Anybody who has ever smelled the filth taken from the bottom of the Hudson, East or Harlem rivers must appreciate the necessity of this improvement.

The project has now assumed such enormous property values and would be of such infinite benefit to the city, state and country in general that it could be carried out as a public enterprise, for the benefit of the people—and in such a way as to wipe out the debt of both the city and the state.

Some claim that the "tidal-prism theory" would be serious. But to show how little harm the tide would do I might say that in a very careful study of years which Mr. H. DeB. Parsons summed up in a valuable paper before the American Society of Civil Engineers he gives the flow through the Narrows as 76,000 cu. ft. per second. Now, during the flood of March, 1913, the discharge of the Mohawk River alone into the Hudson was over 96,000 cu. ft. per second, and the flow of the Hudson below Albany must have been considerably more than 230,000 cu. ft. per second, which shows that the flood flow of the Hudson must be about three times the ordinary flow through the Narrows—and that without attracting any attention to speak of in New York City.

In short, the ordinary daily fluctuations of water passing the Narrows would be much greater than the displacement caused by this plan.

The consummation of this project would mean that all land within 25 miles (at least) of City Hall would be part of New York City, the population of which would be 20,000,000 in a very short time.

The most vivid imagination in the world would have difficulty in foreseeing the many benefits of this undertaking.

F. Kennard Thomson.
New York.

Friction Head in Backwater Problems

Dear Sir: In your issue of May 10 there appeared a brief review of Mr. Kennison's diagrams for solving both Kutter's and Bazin's formulæ for the (uniform) motion of water in open channels. These diagrams are, without doubt, the most convenient yet prepared for the purpose and certainly a vote of thanks is due to their author from all hydraulic engineers. Your review also mentions a method suggested by Mr. Kennison for dealing with the slope of the backwater curve at a point above a dam. Mr. Kennison gives a numerical illustration of the method.

In connection with this method the present writer has been interested in investigating the degree of approximation involved in the assumption evidently made by Mr. Kennison (and also made on page 861 of the American Civil Engineers' Pocket Book, first edition, that in a short reach or stretch of river, within the backwater caused by a dam, the fall of the surface is equal to the friction-head (as based on the mean hydraulic radius and mean velocity for the whole reach). This assumption means that the reach is considered as presenting a case of "uniform motion" (equal sectional areas at the two ends) instead of one of non-uniform motion, happily styled by Professor Merriman "steady non-uniform motion." Some of your readers may perhaps be interested in this investigation.

While the amount of error thus caused may be negligible in many cases the matter is certainly worth looking into, and it is the object of this letter to show to what extent the solution of the example presented by Mr. Kennison is affected by the assumption made.

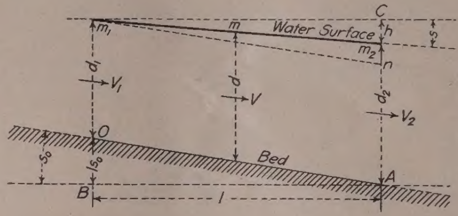
If we consider for a moment the corresponding case in pipe flow it is well known that the fall of the piezometer column from the entrance of a converging Venturi meter tube to the throat is made up almost wholly of the "acceleration head," as it may be called—that is, the difference between the velocity head at the narrow section, or "throat" and that at the entrance section, the friction-head constituting a very small portion of this fall. The assumption in such a case that the whole fall is equal to the friction head would, of course, involve an error of enormous proportions. The corresponding assumption in dealing with non-uniform motion (cross-sections different, but flow steady) in an open channel, involving very gradual changes of section, is not so serious, but might produce considerable error in some cases. Let us now apply the standard relations of hydraulics, as found in the textbooks, to Mr. Kennison's numerical example of a short stretch of river situated upstream from a weir, the flow being steady and the discharge Q ft.-sec. The figure shows a longitudinal section of this reach, which is supposed short that the curve of the surface may be considered straight. It extends from section m_1 to section m_2 where the depth d_2 is greater than that, d_1, at m_1, and the mean velocity V_2 less than that, V_1, for section m_1. Let V be an average mean velocity, also R the average hydraulic radius, for the whole reach, these being practically equal to those, respectively, of the mid-section m where the depth is d. The actual fall of the surface is m_2C, or h. The increase of depth is $m_2n = d_2 - d_1$, while l is the length of the reach. Let h_f be the friction-head, (fl/R) $(V^2/2g)$ for the whole reach, f being the "friction-factor," or "coefficient of fluid friction," proper for the degree of roughness and value of R. Then from the relations of elementary hydraulics we have for this case of "steady non-uniform flow"

$$h = h_f + \left[\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right] \qquad (1)$$

which, if the velocity function in the bracket be called h_{acc} or "acceleration-head" (it is negative in the present case of increasing depth) may also be written

$$h = h_f + h_{acc} \qquad (2)$$

Only in the case of "uniform motion" would h_{acc} be zero and h would then be equal to h_f alone. What is now to be inquired into is the amount of error resulting from neglecting h_{acc} in (1); i.e., of writing $h = h_f$ for this case of "non-uniform" motion. Since



$$h_f = \frac{fl}{R}\cdot\frac{V^2}{2g}, \text{ or } V = \sqrt{\frac{2g}{f}}\cdot\sqrt{R.\cdot\frac{h_f}{l}} = \sqrt{CRs_f}$$

where C is Kutter's co-efficient (or Bazin's) and s_f denotes h_f/l or the "friction-slope," we may also write (1) in the form

(actual slope) $= s =$ (friction slope $+ h_{acc}/l$ that is

$$h/l = s = s_f + h_{acc}/l \qquad (3)$$

$$s = s_f + s_{acc} \qquad (4)$$

adopting the symbol s_{acc} *for* h_{acc}/l and calling it the "acceleration slope."

With s_0 to represent the slope of the bed, and noting that in the figure mn has been drawn parallel to the bed OA, we have, evidently,

$$h = OB + d_1 - d_2 = l\,s_0 + d_1 - d_2 \qquad (5)$$

or

$$d_2 - d_1 = l\,(s_0 - s) \qquad (6)$$

Rewriting (3), i.e., (1), in the form

$$s = s_f - \frac{V_1^2}{2g}\cdot\frac{1}{l}\left[1 - \frac{V_2^2}{V_1^2}\right] \qquad (7)$$

let us make the example more specific by assuming all cross sections of the reach to be rectangular, with a common width of b feet.

The quantity in the bracket may then be transformed thus (since $Q = V_1bd_1 = V_2bd_2$),

$$1 - \frac{V_2^2}{V_1^2} = 1 - \frac{d_1^2}{d_2^2} = \frac{d_2^2 - d_1^2}{d_2^2}$$

$$= \frac{l(d_2 + d_1)}{d_2^2}\cdot\frac{d_2 - d_1}{l}; \qquad (8)$$

or (see (6)),

$$\left[1 - \frac{V_2^2}{V_1^2}\right] = \frac{l(d_2 + d_1)}{d_2^2}\,(s_0 - s) \qquad (9)$$

We now substitute from (9) in (7); and then conceive the sections at m_1 and m_2 to be taken nearer and nearer to m until they finally coincide with it. This coincidence will make $d_1 = d$, $d_2 = d$, and $V_1 = V$; also s becomes the slope of the tangent line to the backwater curve at point m; while s_f becomes the "friction slope" for the same point (and may be taken from a diagram if V, R, and degree of roughness are given).

In this way (7) becomes

$$s = s_f - \frac{2}{d}\cdot\frac{V^2}{2g}(s_0 - s); \qquad (10)$$

which from the nature of its derivation is mathematically exact, but applies, of course, only to the point m of the backwater curve.

For brevity let us denote the ratio $(2/d)$ $(V^2/2g)$ by K, and then solve (10) for the slope, s, of backwater curve at point m; obtaining

$$s = \frac{s_f - Ks_0}{1 - K} \qquad (11)$$

Evidently, from a comparison of (4) and (10), the quantity $[-K(s_0 - s)]$ is the "acceleration-slope," s_{acc} for point m (the assumption that $s_{acc} = 0$ would strictly carry with it the relation $s = s_0$).

Now the numerical data of Mr. Kennison's illustrative example are as follows: For a certain stretch of river (in which before the erection of a weir the motion was uniform, with hydraulic radius $R_0 = 4$ ft. and mean velocity $V_0 = 5.2$ ft. per second the mean sectional area after erection of weir is $F = 2350$ sq. ft. and mean hydraulic radius $R = 12.5$ ft.; while the discharge is $Q = 3340$ cu. ft. per second (as before erection of weir). Evidently $V = Q/F = 1.42$ ft. per second. It is desired to find the new slope, s, for this reach of the river. The slope of the bed (equal to original slope of river surface) is given as 2.35 ft. per thousand feet, that is $s_0 = 0.00235$; while, by a preliminary application of the diagram to the data of the original uniform motion, Kutter's coefficient for roughness is inferred to be 0.035. Mr. Kennison uses the diagram for the data R, V, and n; and finds a slope of 0.000025, which is announced as the new slope of surface for this reach of the river. This value, however, is simply the "friction slope," s_f, upon which the new slope depends only in part; see (11).

Let us now apply (11) to find s, first computing the depth d, the cross-section being rectangular (as already assumed). From R (= 12.5 ft.) and F (the given area, 2350 sq. ft.) we find the wetted perimeter to be $F/R = 188$ ft.; and the two relations $b + 2d = 188$ and $bd = 2350$ enable us readily to determine $b = 158.3$ ft. and $d = 14.83$ ft. Then the ratio K is found to be

$$0.004223\left(= \frac{2}{14.83}\cdot\frac{(1.42)^2}{64.4}\right)$$

while for s_f Mr. Kennison's result, 0.000025, will be taken. Hence (11) gives

$$s = \frac{0.000025 - 0.004223 \times 0.00235}{1 - 0.004223}$$

$$= \frac{0.000025 - 0.000009924}{0.9958};$$

i.e., $s = \dfrac{0.00001507}{0.9958} = .0000151$, or 0.0151 ft. per thousand; whereas s_f is considerably larger, viz., 0.0250 ft. per thousand.

The excess of s_f over s is seen to be 0.0099 ft. per thousand; so that the relative excess is 0.0099/0.0151, = 0.655. In other words, the assumption that $s = s_f$ involves in this numerical instance an error of some 65 per cent.

(If the sides of the reach in question were not parallel, the value of K would be different.)

In two other examples that the writer has tested in which the slope of the bed was considerably less than in Mr. Kennison's, and the increase of depth above the original relatively much smaller, the errors found due to putting $s = s_f$ were found to be some 11 and 16 per cent, respectively. In any given case it would seem best, therefore, to substitute in (11) just as it stands, using the diagram for quick determination of s_f, without attempting to make approximations involving errors of uncertain amount.

When such large errors as the above may arise as the result of making an assumption erroneous in theory the writer hopes to escape the accusation of being hypercritical in calling attention to their possibility.

Ithaca, N. Y. I. P. Church.